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REPORT OF THE COMMITTEE ON THE CAUSE OF THE FAILURE OF THE SOUTH FORK DAM.

WITH DISCUSSION.

To the American Society of Civil Engineers:

The Committee appointed June 5th, 1889, to visit the dam of the South Fork or Western Reservoir, in Croyle Township, Cambria County, and investigate the cause of its failure, following the great rain of May 30th and 31st, 1889, visited the dam as soon as practicable after the re-establishment of railway communication with the locality.

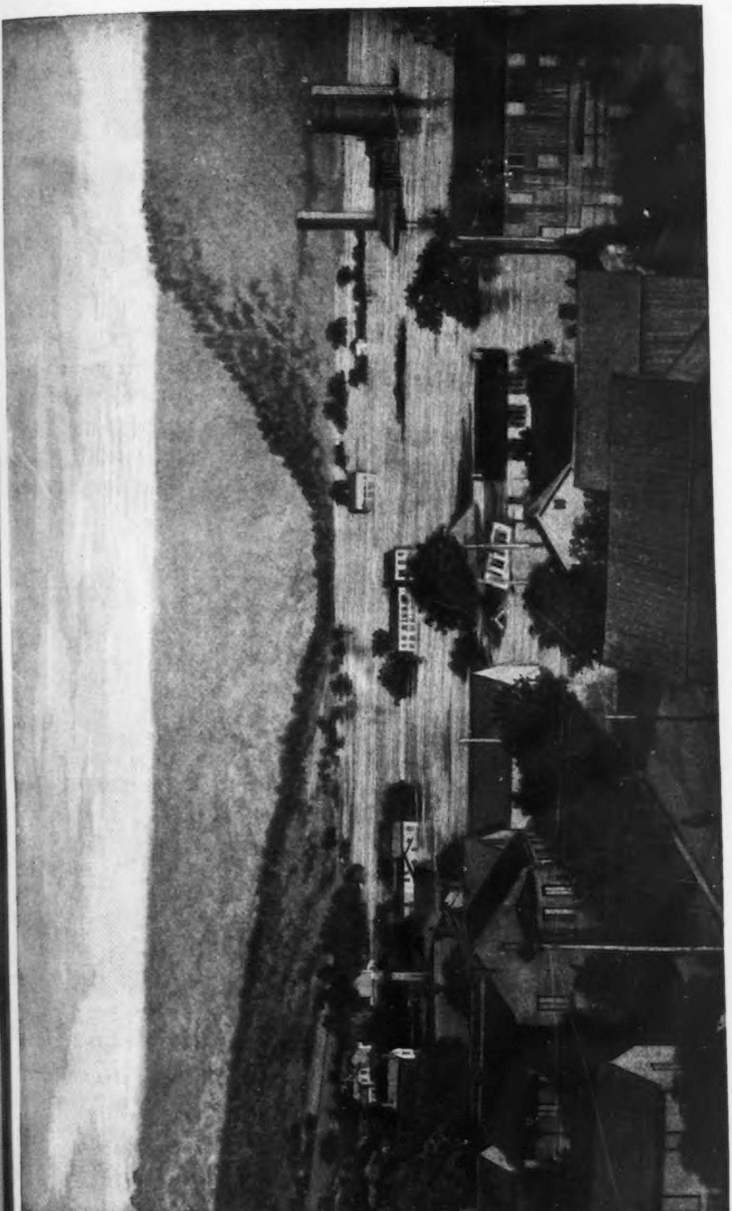
On their passage up the eastern slope of the Alleghany Mountains they observed many evidences of the magnitude of the flood, particularly in the destruction of bridges. Among these were three large bridges over the Juniata River, on the main line of the Pennsylvania Railroad, between Tyrone and Lewistown, besides a great many highway bridges. Four large railroad bridges over the west branch of the Susquehanna River were carried away between Lockhaven and Sunbury on the line of the Philadelphia and Erie Railroad, and many smaller structures were destroyed or injured by the flood. These were undoubtedly expected to provide safely for the passage of the greatest volumes of water of which there

was any experience, and the destruction of so many of them is evidence of the extraordinary character of the flood. The destruction of one of the bridges on the Juniata seemed due to an accumulation of drift, trees, stumps and brush above it, closing the waterways and damming up the flood.

Plates XXXVI, XXXVII and XXXVIII are from photographs of the wreck of the bridge of the Pennsylvania Railroad over the Juniata River, at Lewistown, which is about 45 miles from the nearest part of the summit.

The easterly boundary of the 48.6 square miles of water-shed supplying the South Fork Reservoir is the summit of the Alleghany Mountains. The rain-fall was much greater on the easterly than on the westerly slope. By Plate XLIX and Table No. 1, it appears that on the easterly half of the circle of 100 miles in diameter, of which the middle of this water-shed is the center, the observations indicate an average rain-fall of 6.64 inches, and on the westerly half 2.76 inches, and it is obvious, that except for the failure of the South Fork Reservoir Dam, the damage caused by the flood would have been less on the western than on the eastern slope.

Subsequently to their visit to the dam, the Committee viewed the line of the Pennsylvania Railroad from South Fork Station to Johnstown, a distance of about 9 miles. It follows the Conemaugh River, which flows in a deep valley, the average descent is about 33 feet per mile, the maximum grade being 52.8 feet per mile; but the river, being more circuitous, is about 3 miles longer, its descent between the same points being substantially the same, or about 300 feet. The rise of the water during the flood was generally about 30 feet; at some points, however, it was considerably more. The flood washed away all the bridges between South Fork Station and Johnstown, except the stone bridge near the Cambria Iron Works, and nearly all the embankments and a great part of the tracks. The greatest destruction was within about 5 miles of Johnstown. The streams on both sides of the mountain range had risen very rapidly during the night between Thursday and Friday, but the waters on the eastern slope had reached their flood height earlier on Friday morning than those on the western slope. Nearly the entire working force of the Western Division of the Pennsylvania Railroad had therefore been sent to the Middle Division, and were scattered along the Juniata and Susquehanna Rivers, when the breaking of the South Fork



HIGHEST WATER IN LEWISTOWN. RIVER TO EXTREME RIGHT.

PLATE XXXVII.
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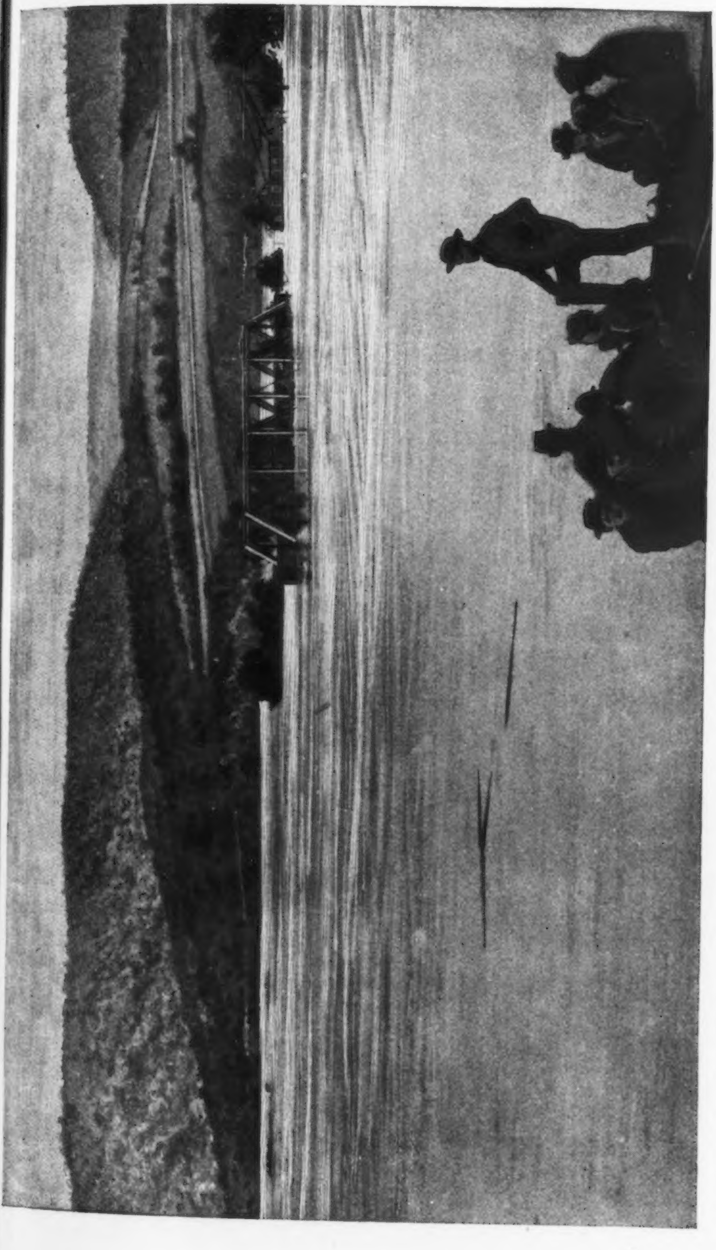

BRIDGE AT LEWISTOWN, (LEWISTOWN DIVISION, PA. R. R.), TAKEN AT HIGH WATER AFTER THREE SPANS HAD FALLEN.

PLATE XXXVIII.
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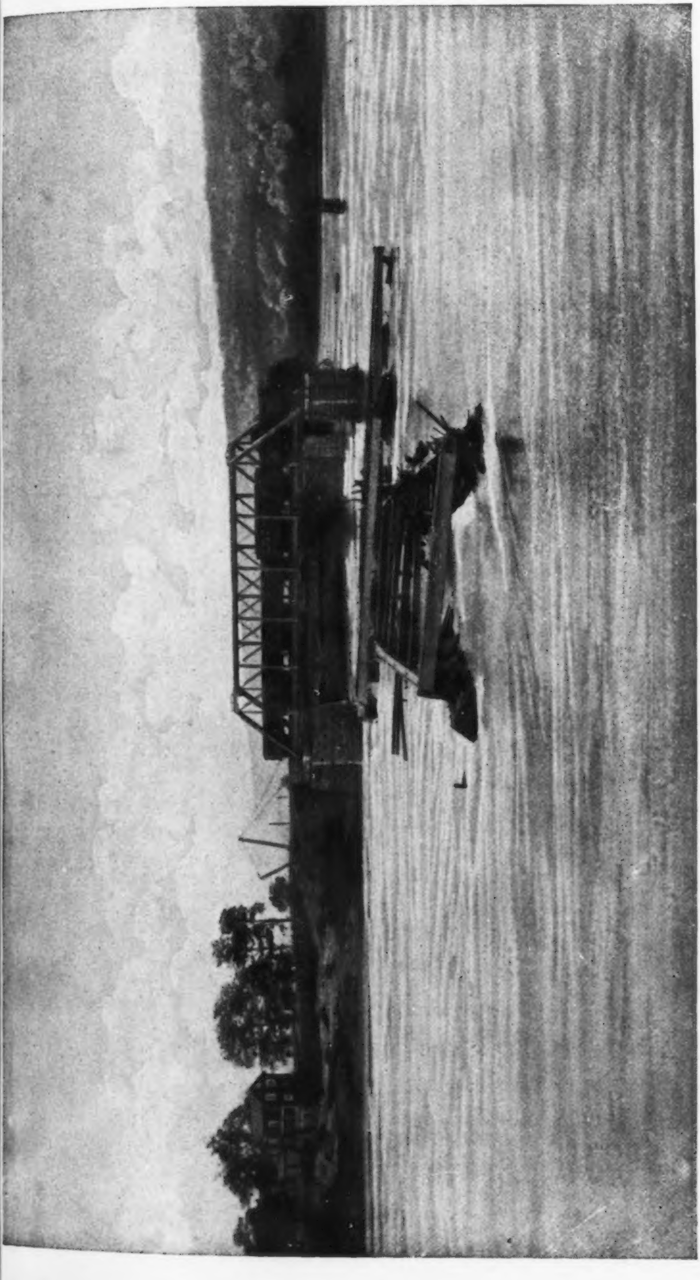

BRIDGE AT LEWISTOWN (LEWISTOWN DIVISION, PA. R. R.), SHOWING WRECK AFTER THE SUBSIDENCE OF THE WATER.

PLATE XXXIX.
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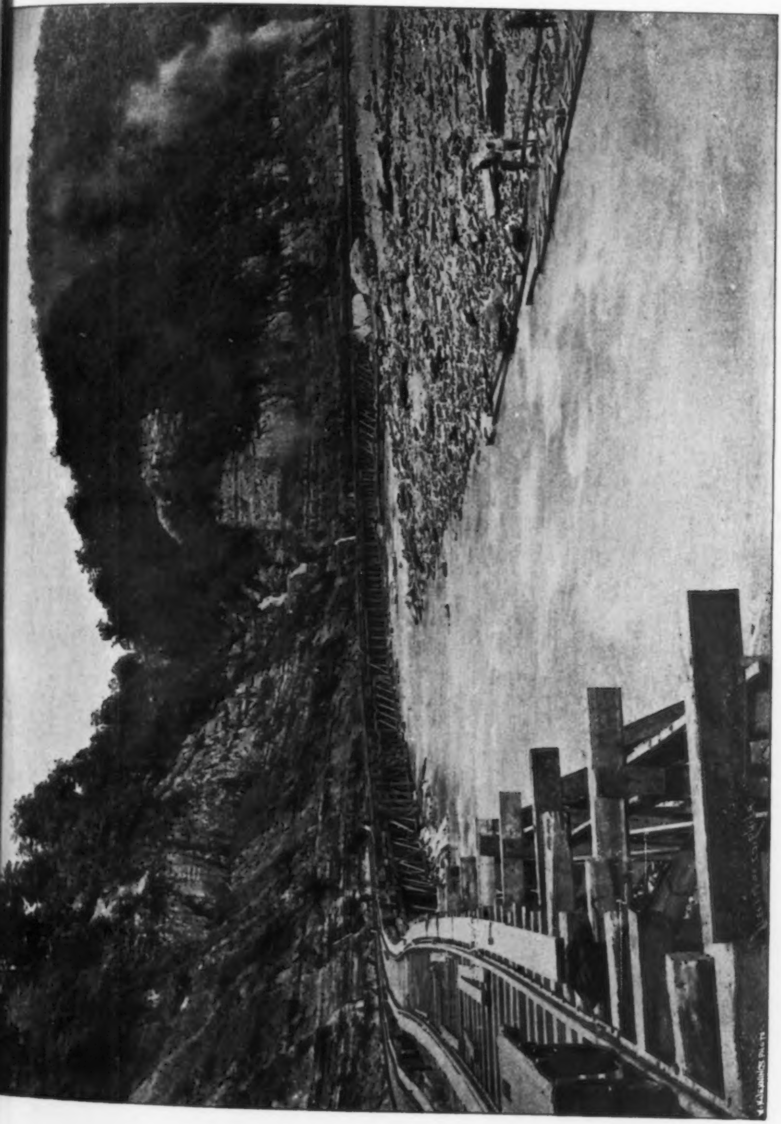

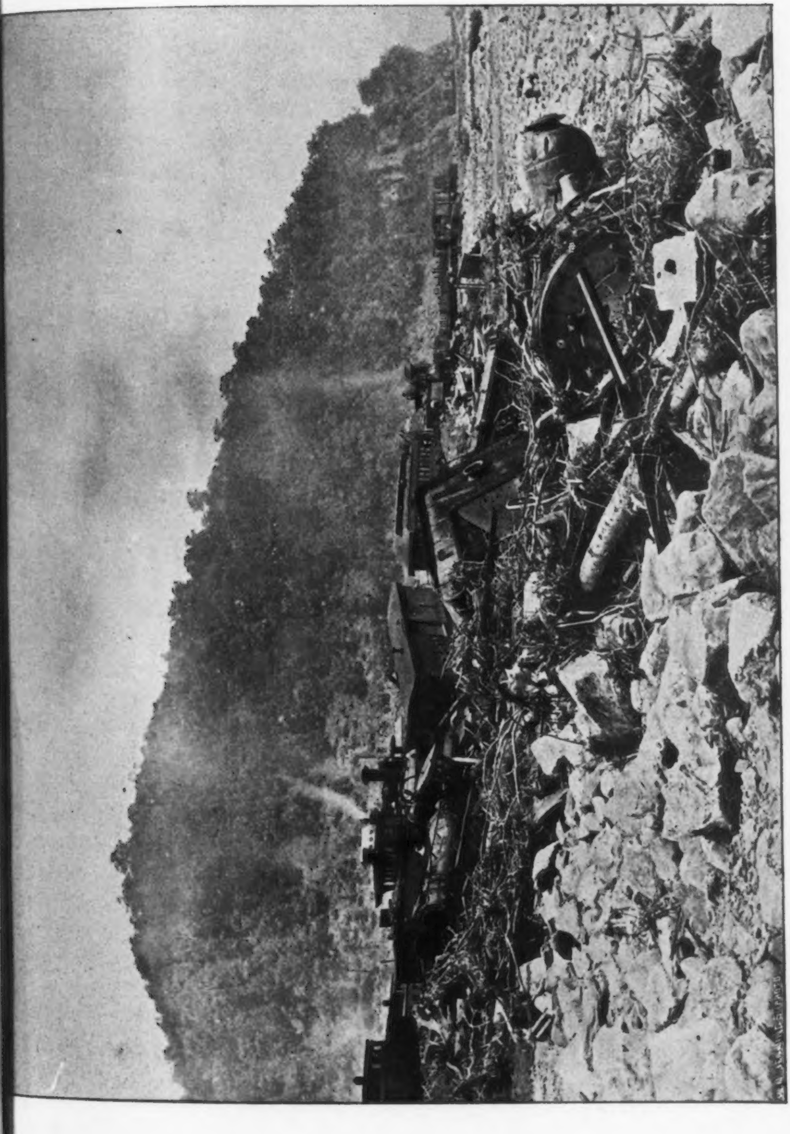

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Reservoir Dam on Friday afternoon suddenly caused the destruction of the railroad in the Conemaugh Valley, with a fearful loss of human life and the annihilation of private property amounting in value to millions of dollars. All means of communication between the inundated sections of the country had been destroyed, and the only reliable line of access for immediate relief to the flooded districts in the vicinity of Johnstown was from the west. When on Saturday morning the full extent of the disaster became known, prompt measures were at once adopted by the public authorities and voluntary organizations for the relief of the sufferers and for substantial assistance to the distressed.

The early restoration of the railroad and the re-establishment of communication were among the necessities most urgently felt. The officers of the Pennsylvania lines west of Pittsburgh were called upon during Saturday for men and materials, and on Sunday night they reported at Johnstown for duty with 1 250 men, fully equipped with tools, provisions, materials and supplies; and from Monday morning, June 3d, these men worked day and night, through rain and flood, until Friday, June 14th, when they met, at Bridge No. 6, $4\frac{7}{10}$ miles above the starting point at the Stone Bridge, in Johnstown, the regular forces of the Pennsylvania Railroad working from the east westwardly. During these eleven days the western forces had built 3 200 lineal feet of trestle work, from 15 to 50 feet in height, laid 29 000 lineal feet of main track, and 9 000 lineal feet of siding and yard tracks, with numerous switches and connections, requiring heavy grading in many places. Plate XXXIX is from a photograph of the Buttermilk Falls trestle, ½ mile east of Conemaugh Station, 2 200 feet in length, double track, which was completed in forty-eight hours.

The New York and Chicago Limited Express, which left Pittsburgh on the day of the flood at 7.15 A.M., had reached South Fork Station, when it received orders at the telegraph tower, just west of the mouth of South Fork Creek, to stop on account of some damage done to the tracks east of South Fork, in the vicinity of Lilly, on the western slope of the Alleghany Mountains. At the same time the Day Express, which left Pittsburgh on the same day at 8.10 A.M., had proceeded to the station at Conemaugh, when it received orders to wait for the reasons given above. The Limited Express was standing at the telegraph tower at South Fork when the dam broke, and the conductor and engineer of the train seeing the volume of water pouring down upon them from South

Fork Creek, disobeyed orders and proceeded with their train, as fast as they could, across the bridge and on eastwardly to high ground. By doing so, they saved themselves from certain destruction, which would have followed had they remained at their posts, as the bridge was bodily swept away and the whole force of the flood rushed against the embankment of the railroad, tearing away the telegraph tower, and carrying the plate-girders composing the bridge over the main Conemaugh River some 300 or 400 feet up stream. The Day Express stood in two sections on the tracks at Conemaugh Station, opposite the round-house, when the flood came upon them. Some of the passengers had left the train before and had gone into the town; but a number were in the train during the flood, which rose to the height of the windows in the cars and the boilers of the engines, burying them with driftwood and debris. The passengers that were lost from the Day Express, which were twenty-eight in number, were drowned in the attempt to save themselves by running from the train to the higher ground on the north side of the tracks. Plate XL is from a photograph of the wrecked train.

We are informed by the Secretary of the Flood Relief Commission that their "records to January 11th, 1890, show that there are known to be lost in the Conemaugh Valley by the flood of May 31st last, 2 280 persons." Of these, 1 675 bodies have been recovered, 1 021 of which have been identified. Bodies are recovered, from time to time, as excavations are made in the debris, and it is possible that a number of persons may have been drowned of whom there is no trace, as there was a considerable floating population in the valley, but from the best information we can obtain we think the total loss of life by the flood did not exceed 2 500.* We have found it impracticable to obtain definite information of the loss of property caused by the flood, but as the result of

* Since the above was written the report of the Secretary of the Flood Relief Commission dated June 30th, 1890, has been received, from which the following is an extract:

"The location being unfavorable for easy escape from the waters let loose from the dam above, the difficulty was greatly increased by reason of a general inundation which existed prior to the breaking of the dam. Nearly the whole city was already submerged to a depth of from 7 to 10 feet, and even had timely warning been given of the impending danger, the result would have been but little changed, as escape through the streets was practically, by reason of the high water, cut off. The most careful investigation, continued to the present time, shows the number to have been 2 142. That is, it is known that persons to this number were in Johnstown at the time of the flood who have not been heard of since as being alive. Some of these may be living and may yet be discovered, but the number given above is not likely to be much reduced. It is probable, too, that the number lost is slightly more than this, but as the additional number could only be composed of persons temporarily in the city, and who had not formed acquaintances or become so identified with a neighborhood as to be missed, it necessarily cannot be large."

recent inquiries of persons in official positions and having the best opportunities of knowing, we estimate the loss by individuals and corporations as between three and four millions of dollars.

At the South Fork Reservoir we examined the remains of the dam, made a survey of the same and its surroundings, obtained photographs of it and of the upper part of the wasteway, copies of which are given in Plates XLI to XLVI. We also made inquiries of persons living in the vicinity and others acquainted with the dam, some of whom witnessed the disaster. Subsequently we have gathered from others, by personal inquiry and correspondence, information as to the rain-fall during the storm which led to the disaster, and as to the construction of the dam and its history. We are under great obligations to Messrs. T. T. Wierman and Antes Snyder for researches in the State records and published documents which have furnished the material for the historical part of this report.

We would call attention to the earth layers of which the dam was composed, which are shown so plainly in Plates XLII and XLIII on each side of the break, also to the abruptness of the banks over which the water fell after the rock backing had washed out, as evidences of the solidity of the earth-work, and that the failure was not due to any leak through this part, but that it was from the overflow of the crest, as testified to by observers.

In Plate XLIV the masses of rock shown were portions of the dam which were carried down by the rush of water, but there is evidence that many were brought back to their present position by the strong eddies on each side of the torrent. This is especially remarkable on the west side where the broken trees along the banks show the direction of the flow, and a high reef is formed nearly up to the dam.

The Commonwealth of Pennsylvania in 1834 had constructed and was operating a system of communication from Philadelphia to Pittsburgh, consisting partly of railway and partly of canal, viz.:

Railway from Philadelphia to Columbia	82 miles.
Canal from Columbia to Hollidaysburgh	172 "
Portage Railway over the Alleghany Mountains from Hollidaysburgh to Johnstown	36 "
Canal from Johnstown to Pittsburgh	104 "
Total railway and canal	394 "

The Canal Commissioners, in their report for 1834, call the attention of the Legislature to the probable failure of the streams to furnish water for the canal during the dry seasons, and caused surveys to be made in order to determine the site of a storage reservoir for the western division of the canal, extending from Johnstown to Pittsburgh. These surveys were made under the direction of Mr. Sylvester Welsh, the principal engineer of the line, during 1834 and 1835. In November, 1835, Mr. Welsh reported a very favorable site for the proposed reservoir, on the south fork of the Little Conemaugh River, and proposed to construct a dam 840 feet long across the valley, which he estimated would create a pool containing about 485 000 000 cubic feet of water. Mr. Welsh proposed a wasteway "at one or both ends of the dam, of sufficient size to discharge the waste water during freshets, and sluices to regulate the supply for the canal." Mr. Welsh estimated that the supply from this reservoir, in addition to the water that flows naturally into these rivers, would supply the water required for the passing of two hundred boats per day, for one hundred and thirty days, without any augmentation from rain.

In April, 1835, the Legislature appropriated $100 000 for various purposes on the public works, among which was mentioned, "The commencement of a reservoir near Johnstown." Under this act Mr. Welsh made further surveys and explorations on Stony Creek and Conemaugh River and their branches, and in his report on the same concludes by recommending a reservoir at the same point as that indicated in his former report. He proposes in this report, a reservoir to cover an area of 465 acres with a content of 525 000 000 cubic feet of water. The plans accompanying the report cannot now be found, but it is gathered from various hints that he proposed to so construct the dam as to give 72 feet depth of water above the bottom of the feeding pipes. In describing the dam, he says: "It is proposed to build the dam of embankment and a wall of masonry in conformity to the plan heretofore submitted. The waste water would be carried through a channel

PLATE XLI.
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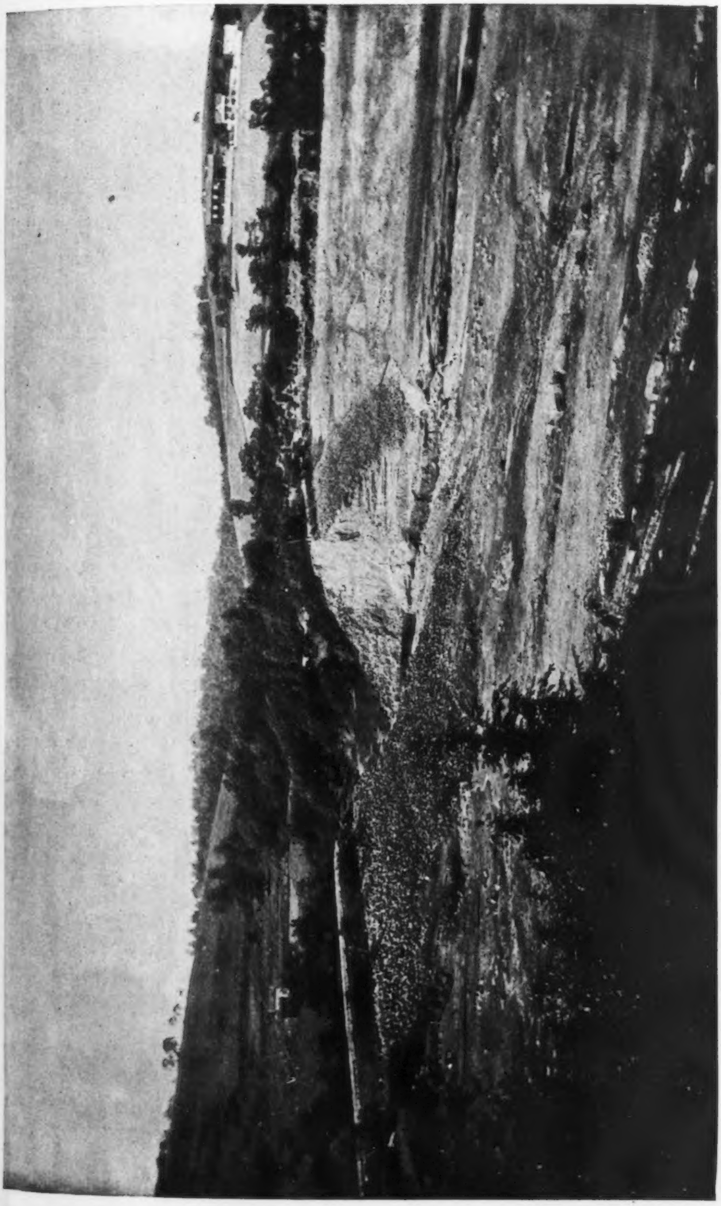
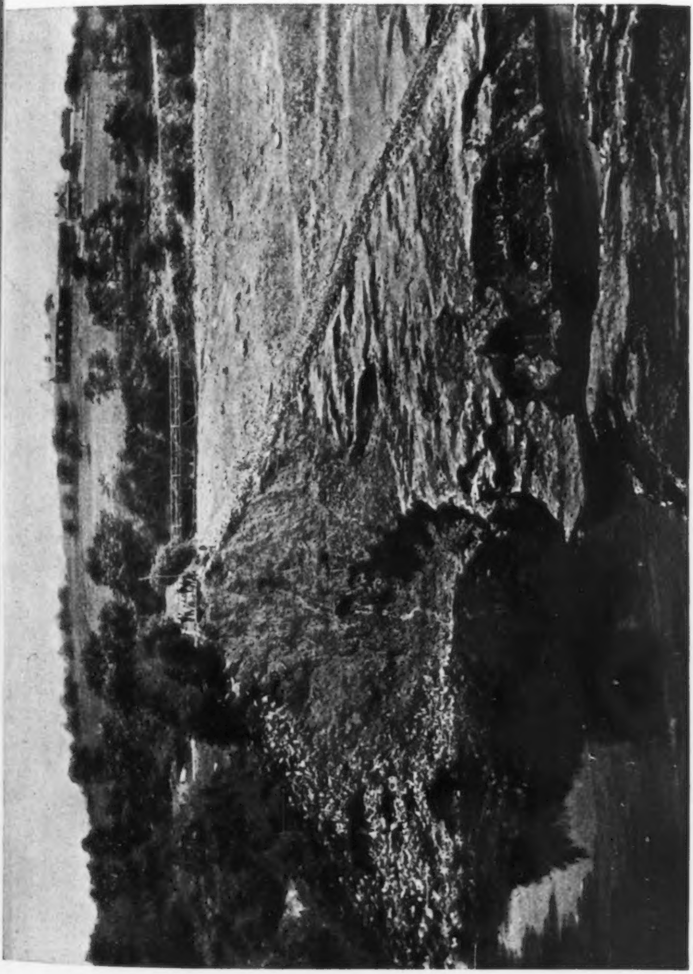


(L. P. de Luze, Phot.) GENERAL VIEW OF THE BROKEN DAM, LOOKING DIAGONALLY ACROSS DAM.



VIEW OF THE BREAK IN THE DAM LOOKING EAST.

(L. P. de Luze, Phot.)

PLATE XLIII.
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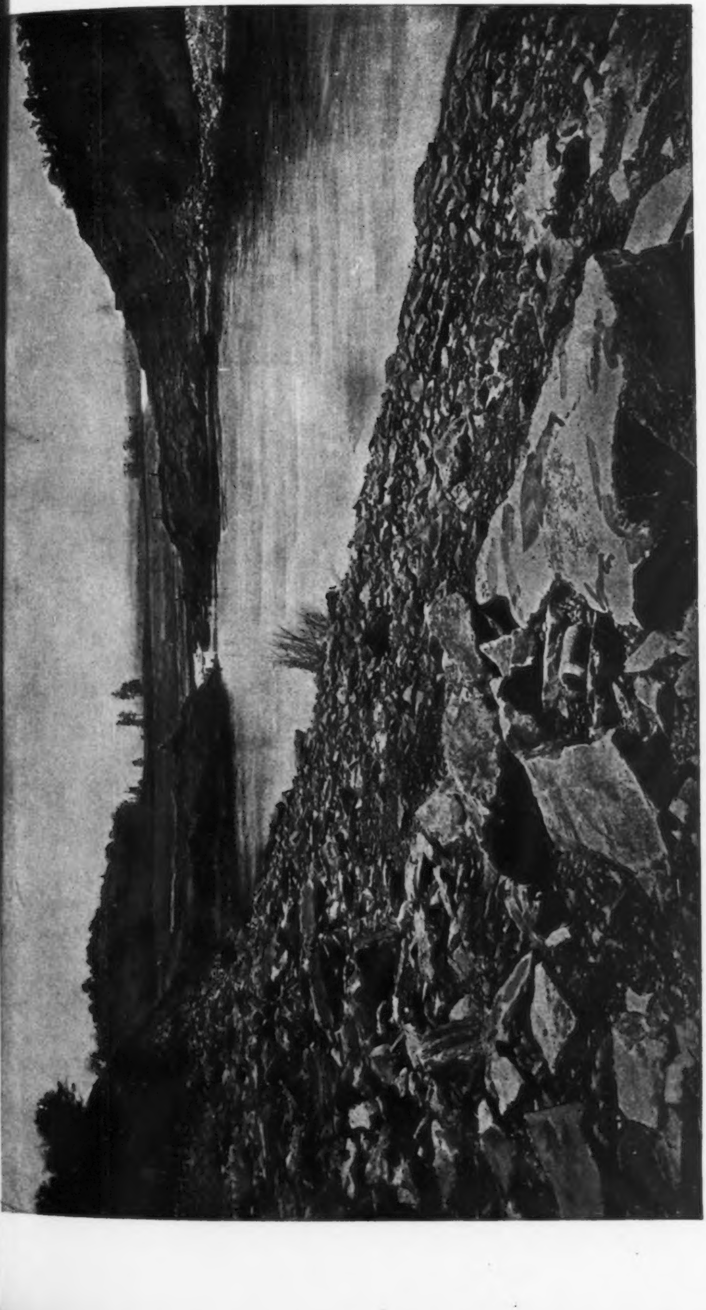
VIEW OF THE BREAK IN THE DAM, LOOKING WEST.

(L. P. de Luze, Phot.)



(L. P. de Luze, Phot.) DISTANT VIEW OF BROKEN DAM, LOOKING SOUTH, AND SHOWING STONES LEFT ON THE SIDES OF THE VALLEY BY THE FLOOD.

PLATE XLV
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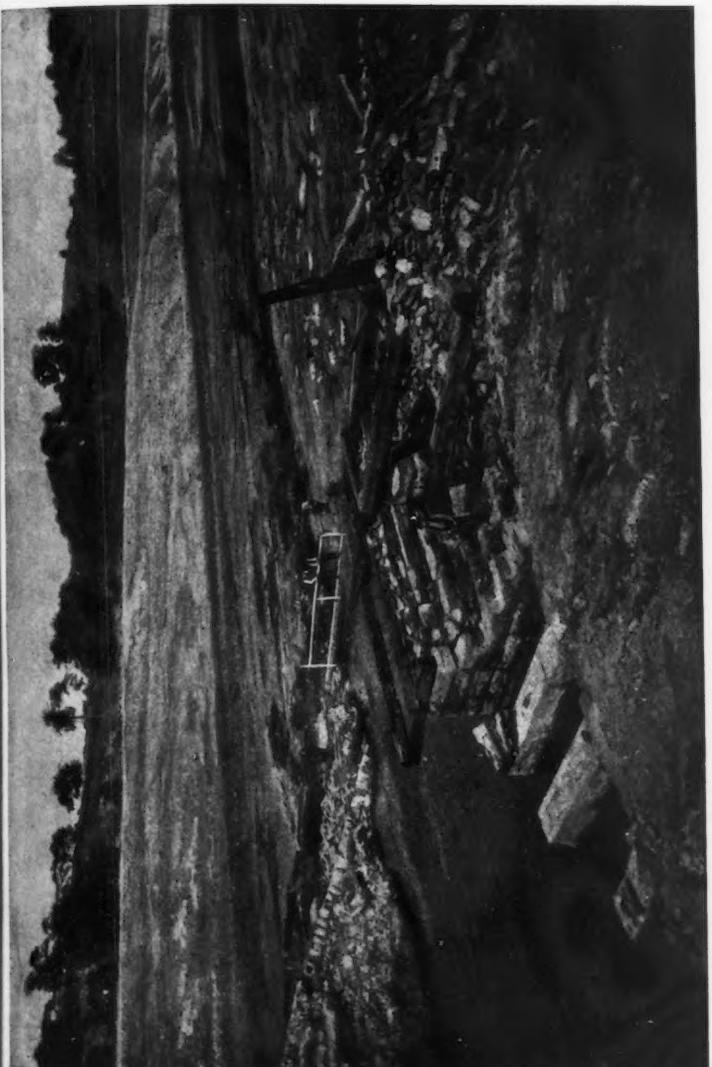

REMNANTS OF GATE CHAMBER IN DAM.

(L. P. de Luze, Phot.)

PLATE XLVI.
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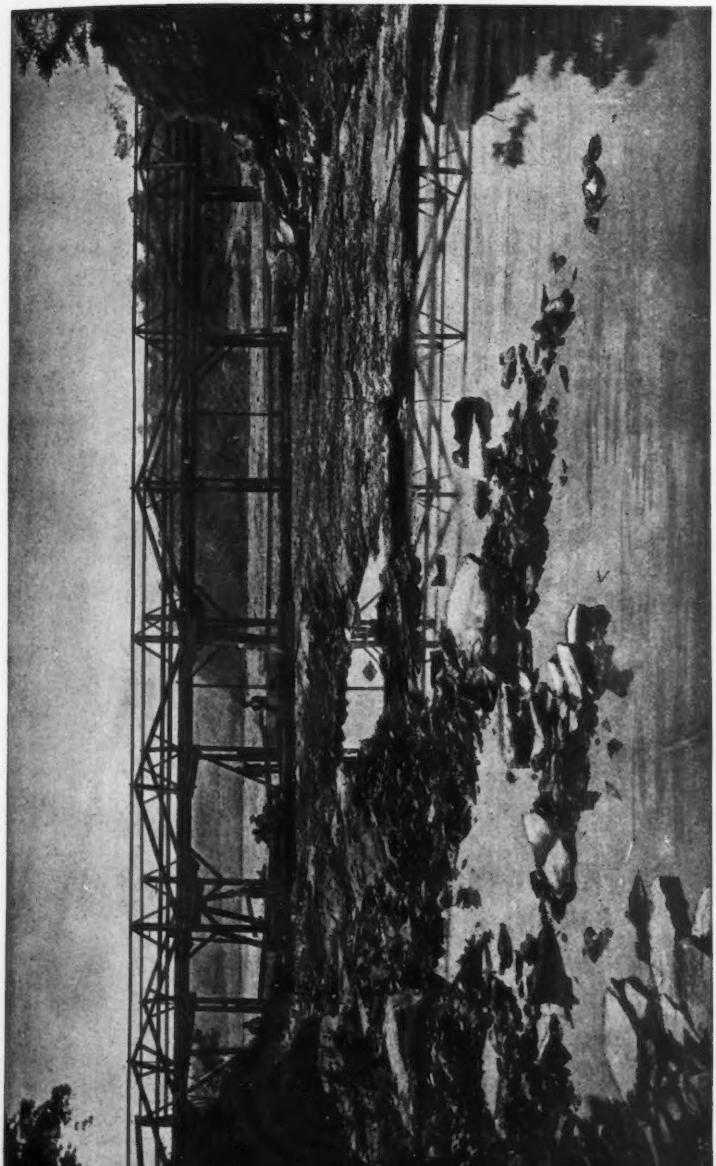

(L. P. de Luze, Phot.) VIEW TAKEN FROM A POINT IN THE WATER-WAY, SHOWING BRIDGE AND FISH SCREENS.

to be made round the end of the dam and passed into the ravine below it. The bed of this channel would be solid rock. No water would be permitted to pass over the dam. The sluice through which the water would be drawn from the reservoir when required for use, should be made of cast-iron set in masonry."

The appropriation of $100 000 for this and other works being all required for the "other works," nothing further was done at this time. In the report of the Canal Commissioners for 1838, in reference to the reservoirs on the eastern and western slopes of the mountains, they conclude by saying: "These reservoirs are absolutely necessary, and should be commenced and finished as soon as practicable." In July, 1839, the Legislature appropriated $70 000 for these reservoirs, and the Board placed the work under charge of William E. Morris,* principal engineer, and in their report for the year ending October 31st, 1839, they say the necessary surveys have been completed, the location made and approved by the Board, and the work placed under contract. In his report of November 1st, 1839, Mr. Morris after approving the location of the reservoir as recommended by Mr. Welsh, discusses the character of the dam to be built, whether "a crib dam with a weir upon the top, or an embankment of stone and earth made perfectly water-tight and raised 10 feet above the surface of the pool, having a wasteway or channel cut in the solid rock, at one or both ends of the dam, for the passage of flood water," and decided in favor of the earth embankment. Mr. Morris' original plan is on file in the Department of Internal Affairs at Harrisburgh, and a copy of the same is given on Plate XLVIII. The execution of the work was submitted to public competition, and the construction of the dam was allotted November 6th, 1839, to James Morehead & Co. This was approved by the Board of Canal Commissioners, December 1st, 1839, and David Watson was elected superintendent of the reservoir,

* Mr. William E. Morris was born at Muncy, Lycoming County, Pa., in 1812. In 1839 he was appointed one of the State engineers of the Western Division of the Pennsylvania Canal, and in August of that year he was sent to Hollidaysburgh to take charge of the construction of the South Fork Reservoir Dam and other works on the western division. In the winter of 1842, on account of financial difficulties, the works were suspended, and he, together with other engineers, was discharged. He was subsequently appointed Chief Engineer and President of the Germantown and Norristown Railroad, in 1853 he was appointed President of the Long Island Railroad, and resided in Brooklyn, and was for several years a vice-president of the Harlem Railroad. He subsequently returned to Germantown, where he employed much of his time as a hydraulic engineer, constructing waterworks for the City of Philadelphia; Tenton, N. J.; Wilmington, Del., etc. He died in Philadelphia, October, 1875.

and was authorized to enter into contract accordingly. The cast and wrought-iron work, and the clearing of the land to be flowed, was let to other parties.

The contract with Messrs. James K. Morehead and H. B. Packer, contractors, is dated January 31st, 1840. The prices to be paid are as follows:

Grubbing and clearing, gross sum....	$1 700 00	
Common excavating above water in puddle ditches..................	15	per yard.
Solid rock excavation above water in puddle ditches..................	50	"
Common excavation below water in puddle ditches..................	34	"
Solid rock excavation below water in puddle ditches..................	1 00	"
Slate excavation in wastes............	32	"
Solid rock excavation in wastes........	34½	"
Embankment of dam, good earth.. ...	25	"
Embankment of dam, coarse stuff.....	30	"
Puddling..............................	38	"
Masonry of sluices....................	6 43	per perch.
Masonry of slope wall................	1 64	"
Masonry rubble......................	3 93	"
Laying pipe.........................	1 00	per lineal foot.

Approximate estimate of work of dam for the Western Division:

Grubbing site of dam, including ground for waste..................................	8	acres.
Clearing ground attached to dam............	5	"
Earth excavation of puddle ditches above water	11 000	yards.
Rock excavation of puddle ditches above water	1 100	"
Earth excavation of puddle ditches below water	2 000	"
Rock excavation of puddle ditches below water	1 000	"
Embankment of dam, good stuff.............	109 000	"
Embankment of dam, slate and stone........	97 000	"
Puddling..................................	2 000	"
Excavation of earth and rock in wastes.......	50 000	"
Vertical wall..............................	4 500	perches.
Slope wall.................................	6 000	"
Masonry in sluices.........................	3 000	"
Laying pipes...............................	350	feet.

SPECIFICATIONS OF THE MANNER OF CONSTRUCTING DAM FOR RESERVOIR, ANNEXED TO AND MAKING PART OF THE CONTRACT.

Grubbing and Clearing.

This term is understood to include the removal and burning up of all trees, stumps, logs, roots, leaves, and vegetable matter whatsoever, from a space 20 feet larger in each direction, than that which will be occupied by the dam and wastes, and also from such other places, from which it may be necessary to procure embankment. And in addition to the above, any trees, that in the opinion of the engineer, may be likely to fall into the wastes or upon the dam, shall be felled and burned. About 5 acres of chopping and clearing next above, and adjoining dam, will be included in the contract for dam.

No part of the aforesaid material shall be thrown into the stream, nor shall it be deposited upon the adjoining land, without the permission of the landholders, in writing, endorsed by the engineer. The whole grubbing must be completed before any embankment is made. The quantity of ground attached to western reservoir is about 13 acres. The quantity of ground attached to eastern reservoir is about 10 acres. Timber, scattering. A gross sum will be bid for grubbing the site of dam and ground attached.

Foundation.

After the grubbing is completed, and previous to commencing the embankment, the foundation shall be prepared by removing from the base of the dam, all sod, vegetable matter, and light, porous material, which shall be deposited in such places as may be directed by the engineer, not less than 100 feet outside and below the slope stakes of the embankment.

Puddle ditches shall then be dug, of such depth and width as may be directed by the engineer, and if necessary, sunk to and into the rock. When the stripping is done, and the puddle ditches dug, the whole area so prepared shall be plowed, with deep and close furrows, parallel to the range of the dam.

The Dam

will be constructed as represented in the plan, with a slope of 2 to 1 on the upper side, and 1½ to 1 on the lower side, it will be raised 10 feet above the water line, and be 10 feet on top. The lower angle will be composed entirely of stone, of such nature as to resist the decomposing action of air, frost and water.

In the outer portion of the stone, for 4 feet thick at top and 20 feet thick at bottom, no stone must be used which does not contain at least 4 cubic feet. The remaining part of the stone may be of any size. Next to the stone will be a body of slate rock or coarse gravel 3 feet thick on top, and 30 feet thick at bottom for western, and 20 feet for eastern reservoir.

The remainder of the bank or dam, being that between the water and the slate, shall be composed of the best water-tight, solid and most imperishable material that can be procured, within ½ mile of the dam. No light, spongy, alluvial, or vegetable matter will be used in its construction. Neither will any coarse gravel or stones larger than 4 inches square be permitted to form any part of it. The whole material of the dam, viz., stone, slate and earth, shall be brought and deposited in the proper place in carts and wagons, and no portion of the dam shall be made by transporting the material in barrows, by schutes, or upon a railway. If it shall be deemed necessary by the engineer, a puddle course of the best fine river gravel, 20 feet in width, shall be carried from bottom of puddle ditch to 4 feet above water line, which said puddle course shall be kept 1 foot higher than the other portions of the embankment, and at all times to be well wet and carted upon, and next the walls, if necessary, well pounded, with a 4-inch rammer. The whole bank shall be made in layers 2 feet thick, be started at the same time, and carried up together, without troughs or hollows, and as nearly level as practicable througout its whole extent. No part of it shall be made in freezing weather. If, during the progress of the work, any part of the embankment, by long exposure or too frequent passage upon it by carts or wagons, shall become so compact upon the surface as to be incapable of uniting completely with the material above to be deposited upon it, such surface shall be well plowed, and, if thought necessary by the engineer, puddle ditches cut, at the expense of the contractor.

In the embankment of the western reservoir, a wall of rubble masonry, made of well-shaped quarried stone, laid in a full bed of cement, with the faces undressed, and the beds and joints close and free from spalls, shall be carried up in the puddle course before-mentioned (or if it be omitted) in the earth embankment. This wall will be started 3 feet below the surface of the rock, if it should be found in excavating puddle ditches, and made completely to fill a trench excavated in the rock for the purpose. It will be 6 feet thick at the bottom, 25 feet high, and 2 feet thick on top; made with buttresses upon each side at intervals of 20 feet, and the difference of thickness between bottom and top, disposed of in offsets of 6 inches in width. No stone shall be used in the wall, of a less size than 2 feet long by 1 foot wide, and 6 inches thick, larger stones to have similar proportions. The wall to be well bound by a system of headers and stretchers so arranged as there shall generally occur one header to two stretchers. The masonry shall be progressed

with as the embankment is raised. Any portion condemned by the engineer shall be immediately taken down and rebuilt.

A slope of wall of dry masonry will be built upon the upper slope of the dam, 15 inches in thickness, backed in by a layer of slate rock or coarse gravel, 6 inches thick. No stone shall be used in its construction which do not reach through the wall, nor any that are of a less size than 4 inches thick by 8 inches wide. This wall shall be neatly laid, the beds of the stone at right angles with the face of the bank, the joints close and free from spalls. A paving of 18 inches depth laid in a similar manner, will cover the top of the embankment.

The cement for the rubble wall and masonry connected with sluices, will be furnished by the Commonwealth, and delivered at the nearest and most convenient point along the line of public improvements, and the contractor will be held responsible for all taken from such places. In working, it shall be mixed while dry with such proportions of clean sharp sand, as the engineer may specify, and the mortar made in small quantities and used immediately. No mortar that has stood over night shall be used in the work. If sand sufficiently clean in its natural state cannot be found, it shall be thoroughly washed.

A waste or waterway will be excavated in the hill at one or both ends of the dam, for the discharge of surplus water in the time of floods, the aggregate width of channels will not be less than 150 feet. The earth covering the rock will first be stripped off, the channel will then be excavated in the rock, leaving for an abutment or guard bank, between the ends of the embankment and the inner slope of the channel, a mass of solid rock, of such width and height, as the engineer may think sufficient. The entrance to the waste or wastes will be as close above the dam as its safety will permit, and its lower termination at least 50 feet beyond the outer slope of the dam.

The material taken from the wastes may be put in its proper place in the embankment of the dam, the stripping of earth upon them will be paid for only as embankment. The slate, detachable rock, and solid rock, found in the wastes will be paid for both as excavation and embankments, except any stone that may be fit for masonry, and used in the walls, which will be paid for only as walls.

Slate rock is such as can be worked with a pick. Detached rock is that which occurs in loose pieces, containing more than 2 cubic feet, and less than 1 cubic yard, or that can be quarried and broken into movable masses without blasting. Solid rock is such as can only be worked by blasting.

Temporary waste weirs, composed of timber and plank, must be constructed by the contractor, at his own expense, at points in the embankment, not more than 15 feet in height above each other, for the purpose of passing with safety to the embankment, any surplus waste water which at any time of high floods, cannot escape through sluices.

Culverts.

The foundations for the culverts or arched way for sluices will be excavated to, and if thought necessary by the engineer, into the rock, which must be suitably leveled for the reception of the walls.

The Masonry

will be range work laid in a full bed of cement mortar, no courses will be less than 8 inches in thickness, nor any stone used in the wall of a less size than 2½ feet long, 16 inches wide and 8 inches thick. The beds and joints of the face stone are to be dressed to a smooth and even bearing 12 inches in width from the face, and brought, when laid, to a joint of not more than ¼ of an inch in thickness. The backing will have the same height as the face stone, the horizontal joints of which shall not exceed ¾ of an inch, nor the vertical joints be at any point more than 2 inches wide, nor shall they average more than 1 inch in width for the length of any joint. All joints shall be broken at least 8 inches.

Both faces of the walls, or the face and back, shall be formed by a regular system of headers and stretchers, laid alternately, and so arranged that the headers upon the front shall be opposite the stretchers on the back.

Both sides of the jambs or abutment walls and both ends of the cross-walls will be considered as face-work. The side walls and ends of the chambers, or entrance of the sluices, both within and without, shall have a rock dress upon the face with a cut draught ¼ inch wide around the edges. Upon the inner sides and ends, the rock dress-work shall not vary from the line or draught more than ½ inch, but upon the outside of the jambs it may be left rough.

The abutment of the part of the culvert below the ends of the pipes, shall be made in the same manner, except the inner face-work shall be picked off to within ½ an inch of variation from the cut draught.

The whole beds or joints of that part of the masonry between the two divisions of the culvert, upon which the pipes are laid, shall be dressed to a smooth even bearing, and so laid as to form joints of not more than ¼ of an inch in thickness. The upper courses shall be 20 inches in thickness and cut to fit the pipes for half their circumference.

If required, a paving shall be placed between the side walls, laid in cement, and in the same manner as the jambs of the culvert. The arch shall be 20 inches in thickness. No stones to be used in its construction, which do not fill the entire depth, are not 18 inches wide, and which do not measure at least 6 inches in thickness on the outside of the arch. They shall be dressed by a pattern to a full bed; the joints not to exceed ¼ of an inch in thickness.

Piling walls or yokes shall be constructed at such points as the

engineer may direct, and the walls of the sluices well united with the vertical wall of dam.

All stone used in the walls must be sound and durable, and approved by the engineer, and materials condemned by him shall immediately be taken to such places as he may direct. That part of the wall exposed to the action of the water, issuing from the pipes, must be made from the hardest and most durable sandstone. Stone more easily dressed may be used in other parts of the work. The mortar will be made as specified for vertical wall of the dam. The lower end of the culvert shall be of neatly cut sand-stone, finished with pilasters and mouldings, as represented in the plan. The parapet and wing walls to be finished with a cut coping of sandstone, 2½ feet in width, securely clamped and leaded. The iron, wrought and cast, the stop-cocks, and fixtures, and lead for joints of pipes, will be furnished by the Commonwealth, and delivered at the site of reservoir, when the contractor will be required to put them in their respective places in the walls.

The wall irons will in all cases be bedded in the stone. The Commonwealth will furnish a force pump to test the pipes, joint by joint, and an experienced machinist to superintend and direct the laying of the pipes, leading the joints, and applying the test, the contractor to put up the force pump and be at all other necessary expenses, for fixtures, hands, etc., to make a secure and perfect job. Any length of pipe or joint that shall prove defective upon the application of a pressure equal to 300 feet head of water shall be taken up and rejected or relaid; proposals will state a price per foot lineal for laying pipes. The culvert walls and pipes to be puddled in such places as the engineer may direct. A suspension way or walk will be constructed from the lower end of the culvert to the stop-cocks at the end of pipes; it will consist of a plankway 2½ feet wide, and 2 inches thick, suspended from the arch of the culvert by iron rods and terminating upon the culvert wing.

A stack of masonry, laid as specified for jambs of culvert, with a rock dress on the faces, will start at the lower ends of chamber as represented in the plan, and be raised to a height equal to that of the dam, finished with a coping of cut sandstone, 12 inches thick and 3 feet wide, securely clamped and leaded; no allowance made for bailing water, coffer dams or other extras.

In the report of the Canal Commissioners of January 1st, 1841, in reference to this reservoir, they state: "It was commenced last spring. The clearing is nearly completed, and the pipe cast and now ready for delivery. All the work at the dam below the surface of the water in the stream, has been done except the sluice walls. If the necessary appro-

priation be made, the work can be completed and brought into use sometime during the next summer." Estimated cost, $188 000.

November 30th, 1841, Mr. W. E. Morris reports: "Since last fall the contractors have steadily pushed on the work at the dam, though, from the smallness of the appropriation, with a moderate force. The sluice walls are raised sufficiently high to receive the pipes, each range of pipe about 80 feet long, has been laid and tested by a head of 300 feet." * * * * "The estimated cost of work done at contract prices is $80 000."

The United States Bank of Pennsylvania, chartered in 1836, with a capital of $35 000 000, intended to take the place of the United States Bank, the renewal of whose charter had been vetoed by the President, finally suspended payment February 4th, 1840. The finances of the State were in such bad condition that the Legislature was forced to curtail its appropriations for the public works to what was absolutely necessary to keep them open for business. April 3d, 1841, an act was passed appropriating $50 000 to pay for all work done on the eastern and western reservoirs up to May 1st, 1841. The contractors were paid up in full and the work stopped. January 3d, 1846, $20 000 was appropriated to the western reservoir, and the Canal Commissioners were required to complete it with as little delay as possible. William E. Morris was called upon to prepare new plans and specifications to be exhibited at a letting on March 3d, 1846. They were the same as those prepared by him in 1839, except that a frame tower was substituted for one of masonry. The former contractors still claimed the right to complete the dam under their contract of 1840. The Canal Commissioners, on account of the large expenditures required to repair the damages by the flood of the spring of 1846, postponed the work, and the appropriation reverted to the general fund and could not be expended without another appropriation. The appearance of cholera in 1848 and its continuance through 1849, caused a general derangement in the business of the country, and it was not until 1850 that a new appropriation of $45 000 was made by the State, and Messrs. Morehead & Packer were permitted to proceed with the work under their original contract. From delay, however, this appropriation reverted to the general fund, and a new appropriation of the same amount was made April 15th, 1851.

The work was commenced May 1st, 1851, and prosecuted as rapidly as practicable. May 4th, 1852, $55 000 was appropriated for the completion of the work, which in the meantime had so far advanced that on June 10th, the sluice gates were closed, and by the end of August water had accumulated in the pool to the height of 40 feet above the feeding pipes. At that time the water in the canal began to fail, and in order to meet the requirements of navigation, the supply in the reservoir was drawn and found ample to keep the canal in full operation until the dry season was over. The embankment of the dam being new, is was deemed

unsafe to fill the reservoir to a greater height than 50 feet above the feeding pipes for the season of 1853, and this season being an uncommonly dry one, the supply of water in the reservoir became exhausted before the close of the season. The contractors seem to have completed the work on the reservoir some time early in the season of 1853. The entire expenditure up to that time, as nearly as can be ascertained, was $166 647.50. During the year 1854, there were two breaks which the Commissioners reported as slight and promptly repaired, but without giving their location or nature.

The canal continued to be operated by the Commonwealth until July, 1857, when it, with the western reservoir, was sold to the Pennsylvania Railroad Company. After it came into their ownership, leaks occurred where the feeding pipes entered the culvert, and finally, in July, 1862, there was a serious break at this point, which washed out the upper end of the culvert and part of the embankment of the dam. About 20 feet of the lower end of the culvert, and the embankment over it remained. The reservoir at this time was only partially filled, and the discharge from the breach passed through the remains of the culvert and was comparatively slow, requiring the greater part of a day to empty the reservoir. The damage caused by it on the stream below, as we are informed, was limited to the washing away of a portion of the embankment of the Pennsylvania Railroad at the eastern end of the bridge over South Fork. In the report of the Chief Engineer of the Pennsylvania Canal to the Pennsylvania Railroad Company, dated January 1st, 1863, it is stated that "the western reservoir dam gave way in July last from a defect in the foundation of the culvert. It is not proposed to repair this work, as an early abandonment of the upper western division is in contemplation."

In the annual report of January, 1864, the Chief Engineer stated that "The upper Western Division, extending from Johnstown to Blairsville, was abandoned as a navigable canal on May 1st last, since which time it has not been kept open." In March, 1875, the property, consisting of about 500 acres of land, including the site of the reservoir, was sold to Mr. John Reilly, who conveyed it to the South Fork Hunting and Fishing Club of Pittsburgh, in June, 1880. In April, 1880, the work of repairing the breach in the dam of July, 1862, was commenced by the Hunting and Fishing Club. It was let out by contract to Colonel B. F. Ruff. The five lines of 24-inch sluice pipes were taken out, the masonry on which they were laid and the remains of the culvert were left. A sheet piling of plank was put across the lower part of the breach. The original plan of making the lower angle of the embankment of stone was adopted, and stone of as large size as could

be obtained in the vicinity were dumped into the breach, letting them form natural slopes. This stone embankment was carried up until it reached such a height as to enable a road to be graded down from the crest of the remaining parts of the dam on each side of the breach, so that material could be hauled in carts from the borrow-pits on the hill side. There being no sluices for the discharge of the flow of the stream, the surplus water found its way through the stone embankment, the water in the reservoir rising as the filling on the upper side of the stone embankment proceeded. The washing of the filling through the stone embankment was prevented by covering its face with brush, hay, etc. The material relied on to form the water-tight embankment consisted of clay and shale, which was dumped in on the upper side of the stone embankment, and carried up in layers to the full width of the remaining parts of the original dam. There was no systematic puddling done, but the hauling by teams over the freshly deposited material, which was kept wet by the rising water, made a fairly compact embankment on the upper side of the stone embankment. The work was not completed that season, and during the following winter it was damaged by a flood, and the next year, 1881, the Hunting and Fishing Club completed it by day work. The slopes on both sides of the embankment were covered with a heavy rip-rap.

According to Mr. Morris' original plan and specification the top of the dam was 10 feet wide and 10 feet above the ordinary surface of the water in the reservoir, which is understood to be nearly the same as the floor of the wasteway. By our levels the floor of the wasteway for 176 feet from the lake averages 1602.82 feet above tidewater. The elevation at eight points on the top of the remaining parts of the dam, where not affected by the late washout is 1610.78 being 7.96 feet above the floor of the wasteway, or 2.04 feet below its height as originally designed. This accords substantially with the statements of parties living in the vicinity, to the effect, that when the breach was repaired in 1880–81, the top of the dam was lowered about 2 feet in order to make a more convenient roadway over it, the remains of which are now 15 to 20 feet wide.

On May 30th, 31st and June 1st, 1889, there was a great rain on an area of about 20 000 square miles, mostly in Pennsylvania, but extend-

Observations of the Rain-fall on Thursday, May 30th, and Friday, May 31st, 1889, at Sta
of the United States Signal Service and the Pennsy

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STATION.		Quarter, and Approximate Direction and Distance of the Station from the Center of the Water-shed.			RAIN-FALL BY UNITED S	
Place.	County.	Quarter.	Direction.	Distance, Miles.	Time of Beginning.	Time of Ending.
Grampian Hills......	Clearfield............	North-east.	N. 6° E.	45	30th, 4.30 P.M.	31st, 11.20 P.M.
Phillipsburgh........	Centre........... ..	"	N. 32° E.	47	30th, 3.50 "	....
Tipton	Blair............. ..	"	N. 46° E.	28	30th, 4.00 "	....
Altoona..............	Blair..................	"	N. 50° E.	21	30th, 3.30 "	....
Hollidaysburgh.......	Blair	"	N. 64° E.	16	30th, 8.00 "	31st, 12.00 P.M.
Petersburgh..........	Huntingdon.........	"	N. 66° E.	38	30th, 3.00 "	31st, 11.50 P.M.
Huntingdon.....	Huntingdon..........	"	N. 74° E.	39	30th, 4.00 "	....
Blue Knob............	Blair..................	"	N. 84° E.	12	....	....
McConnellsburgh	Fulton	South-east.	S. 55° E.	48	....	....
Charlesville.........	Bedford.............	"	S. 24° E.	29	....	....
Somerset............	Somerset............	South-west.	S. 42° W.	28	30th, 10.00 P.M.	31st, 10.00 A.M.
Greensburgh.........	Westmoreland.......	"	West.	42	30th, 6.00 P.M.	....
Johnstown...........	Cambria.............	North-west.	N. 80° W	9	....	....
Saltsburgh...........	Indiana	"	N. 70° W	40	....	....
Indiana..............	Indiana	"	N. 45° W.	32	30th, 2.00 A.M.	31st, 7.30 P.M.

TABLE No. 1.

1889, at Stations within a radius of fifty miles from the center of the Water
the Pennsylvania State Weather Service, as published in their respective Mo

. BY UNITED STATES SIGNAL SERVICE.				RAIN-FALL BY PENNSYLVANIA STATE WEAT		
	Rain Fall.					
Time of Ending.	Thursday, May 30th. Inches.	Friday, May 31st. Inches.	Amount in the two days. Inches.	Time of Beginning.	Time of Ending.	Thursday, May 30th. Inches.
31st, 11.20 P.M.	0.23	8.37	8.60	30th, 11.40 P.M.	31st. 11.20 P.M.	0.23
....	....	2.83	2.83	....	....	...
....	0.80	3.35	4.15	....	....	....
....	....	3.03	3.03	30th, 3.00 P.M.	....	....
31st, 12.00 P.M.	2.11	3.99	6.10	30th, 4.00 P.M.	....	0.39
31st, 11.50 P.M.	0.01	6.60	6.61	....	....	....
....	0.60	4.22	4.82	30th, 4.00 P.M.	....	....
....	....	7.90	7.90	....	....	....
....	1.23	7.08	8.31	30th, 4.00 P.M.	31st, 12.00 P.M.	1.23
....	0.61	6.71	7.32	30th, 3.15 P.M.	....	0.61
31st, 10.00 A.M.	....	4.43	4.43	....	....	....
....	....	1.70	1.70	....	....	....
....	....	....	....	....	....	....
....	....	1.94	1.94	....	....	....
31st, 7.30 P.M.	2.00	1.00	3.00	....	....	....

e Water-shed of the South Fork Reservoir. Collated from the Records
ctive Monthly Reviews for May, 1889.

tate weather service.			Average rain-fall in the two days. Inches.	Remarks.
Rain-fall.				
Thursday, May 30th. Inches.	Friday, May 31st. Inches.	Amount in the two days. Inches.		
0.23	8.37	8.60	8.60	From 11.40 P.M., May 30th, to 11.20 P.M., May 31st, 8.37 inches of rain. Six inches of rain fell in seven hours.
...	2.83	2.83	2.83	
....	....	....	4.15	
....	....	....	3.03	
0.39	5.12	5.51	5.80	From 9 P.M., May 30th, to 9 P.M., May 31st, 5.12 inches of rain
....	....	....	6.61	
....	....	....	4.82	
....	....	....	7.90	
			5.47	Average rain-fall in north-east quarter.
1.23	7.08	8.31	8.31	5.45 inches of rain from 4.00 P.M., May 30th, to 4.00 P.M. May 31st. 8.31 inches of rain in 32 hours.
0.61	6.71	7.32	7.32	
			7.81	Average rain-fall in south-east quarter.
....	....	....	4.43	
....	....	....	1.70	
			3.06	Average rain-fall in south-west quarter.
....	....	....	....	Rain-gauge carried away by the flood at 10.44 A.M., May 31st. Not more than three-fourths of the rain-fall in the Johnstown Valley had fallen up to 3 P.M. on the 31st.
....	....	....	1.94	
....	....	....	3.00	
			2.47	Average rain-fall in north-west quarter.

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ing into Maryland, Virginia and West Virginia. It is estimated that about three-fourths of this area, or 15 000 square miles, constituted the flooded area in Pennsylvania; the greatest amount of water appears to have fallen near the summit of the Alleghany Mountains and a little east of the same, where the rain-fall was reported to have been 8 to 10 inches in eighteen to thirty-six hours. Official records of the rain-fall are reported from about fifty stations, but there is none from any point on the water-shed of the South Fork Reservoir, and it can only be inferred from the observations reported from other water-sheds in the vicinity. To enable this to be done approximately, the observations reported from stations within 50 miles of the center of the water-shed of the South Fork Reservoir (see Plate LIX) are collected in the annexed Table No. 1. As will be seen, the stations are limited to the area of a circle of 100 miles in diameter having its center in the center of the South Fork water-shed, and they are arranged according to the quarter of the circle in which the station is situated. The results are as follows:

In the northeast	quarter,	average	rain-fall....	5.47	inches.
" southeast	"	"	"	7.81	"
" southwest	"	"	"	3.06	"
" northwest	"	"	"	2.47	"

The northeast and southeast quarters include parts of the summits of the Alleghanies and of their eastern slopes. The easterly boundary of the South Fork water-shed is this summit, and by inspection of the table and the accompanying map, we infer that the rain-fall on this water-shed was from 6 to 8 inches. As we are informed, the heaviest part of the rain was during the night of May 30th, and 31st, during which time it appears that there must have been several hours when it fell at a rate of not less than two-thirds of an inch an hour on this water-shed. By the survey of the water-shed above the dam, made for us by Mr. A. Y. Lee, civil engineer of Pittsburgh, its area is 48.6 square miles. (Plate XLVII is a reduced copy of his map.)

Mr. John G. Parke, Jr., civil engineer, had been employed for about two months previous to the disaster in improving the drainage of the property of the South Fork Hunting and Fishing Club, and was residing at their club house situated on the southerly shore of the reservoir about three-quarters of a mile from the dam. Mr. Parke has

kindly furnished us with an account of his observations and experience on the day of the failure; we do not undertake to condense it, but give it in full as the best account we have been able to obtain.

M. J. BECKER, Esq.,

President Am. Soc. of C. E.:

DEAR SIR,—Being requested by your Committee to give an account of the destruction of the South Fork Dam, I will do so by narrating my experience of the affair and the few observations I made at the time.

On the evening preceding the destruction of the dam, to the best of my recollection, we had many evidences of an approaching storm, and when it grew dark, we had a violent wind storm, and the tree tops about the house were moaning and creaking unusually, but no rain fell. About 9 o'clock I had occasion to go out of the house a short distance and noticed that the board walk was wet and that there had been a slight rain-fall, but it was not raining at the time, and the sky was much brighter and evidently clearing off, but there was still a high wind blowing. These sudden, violent wind storms, very often accompanied by heavy but brief rain-falls, were customary in that mountainous country, as I had noticed during my two months' location at the lake previous to this time.

I retired shortly after 9 o'clock and slept very soundly, awaking once towards morning and hearing a heavy rain. When I awoke at about 6.30 on the morning of the 31st, I found it very foggy outside, and on going out, found the lake had risen during the night probably 2 feet, and I heard a terrible roaring as of a cataract at the head of the lake, about a mile above the club house where I was staying. After eating breakfast and returning to the shore I found the lake had risen appreciably during our absence, probably 4 or 5 inches, and with difficulty secured a boat, and with a young man who was employed on some plumbing work at the cottage, I rowed to the head of the lake to see the two streams that were pouring into the lake with such an unusual roar. I found that the upper one-quarter of the lake was thickly covered with debris, logs, slabs from sawmill, plank, etc., but this matter was scarcely moving on the lake, and what movement there was, carried it into an arm or eddy in the lake, caused by the force of the two streams flowing in and forming a stream for a long distance out into the lake. The lake seemed very high when I reached the head, for we were able to row over the top of a four-wire barbed fence which stood near the normal shore line and we rowed for 300 feet across a meadow which was covered with water, for it was very flat and a rise in the lake of a foot, covered a large area. I did not get very near to one stream (the Muddy Run), but could see the volume it was pouring in by its current in the still water, but we did go up the shores of the South Fork Creek and found it widely overrunning its banks. In its normal condition it is

about 75 feet wide and barely 2 feet deep—many places not that deep, but varying as every mountain stream does. But on this day it was a perfect torrent, sweeping through the woods in the most direct course, scarcely following its natural bed, and stripping branches and leaves from the trees 5 and 6 feet from the ground. We tramped through the fields adjoining the woods in which the stream was boiling, for a half mile above its mouth and could appreciate its volume and force, for I was familiar with the region and saw where the stream covered a portion of the township road for a depth of 3 feet, which is never covered except in floods. Returning to our boat, we found it almost adrift, the water having risen during our absence. We rowed to the club house and found the water had risen at a wonderful rate during our row to the top of the lake. I had been thinking of the dam and was not surprised when landing to be told that the water was nearly over the dam and that men and a plow were needed there. So taking a horse from the stable, I rode to the breast and found Colonel Unger, President of the South Fork Fishing and Hunting Club at work on the dam with a number of Italian laborers (that we had employed on some sewerage work); there were about sixteen of them. Half of them were cutting a ditch through the shale rock at one end of the breast. This ditch was cut through the original ground and about 25 feet from the constructed portion of the breast. The shale was so tough that they could not cut it more than about 14 inches deep and about 2 feet wide, but when it was cut through to the lake, the water rushed in and soon made it a swift stream, 25 feet wide and about 20 inches deep, but the rock was so hard that it could not cut it any larger than this. Previous to this being opened and shortly afterward, I made two observations of the height of the water, and the lake in the hour had risen 9 inches. During the digging of this ditch, I rode back and forth over the dam directing the laborers. We had a plow at work throwing up a furrow, and thus raising a temporary barrier or breast to retard the water flowing over the dam, should it reach that height which it was gradually doing. I noticed that the waste-weir proper was discharging to its full capacity, and that there was no drift or other matter to clog it, except a road bridge supported on small posts which were apparently offering but little resistance as the weir was narrower by about 15 feet at 100 feet from its mouth, and this contraction compensated for the resistance to flow offered by the bridge supports. There was probably 7 feet of water in the weir at the time. There were some iron screens between the foot of each post on the outer row of the bridge supports, but they were but 18 inches high and could not have been removed, had we wished to, owing to the depth and velocity of the water. The water in the lake rose until it was passing over the breast, notwithstanding that the lake had then the two outlets (the waste-weir and the one cut by the laborers). The breast was slightly lowered in the center and the water

washed away our temporary embankment thrown up by the plow and shovels, and the water was passing over in many places in a distance of 300 feet about the center of the breast; the men stuck to their task and worked until the water was passing over in nearly one sheet, and then they became frightened and got off the breast. I saw what would be the consequence when the water passed over the breast and rode to South Fork Village and warned the people in the low lands there, and had word telegraphed to Johnstown that the dam was in danger. The people in South Fork heeded the warning and moved out of their houses. When I left South Fork to return it was just twelve o'clock noon, and the water had been flowing over the dam for at least a half hour. I rode back up to the lake 2½ miles through the valley and found the men had torn up a portion of the flooring of the waste-weir bridge and were endeavoring to remove the V-shaped floating drift guard that projected into the lake. It was a light affair and was built to float on the surface of the lake and catch twigs, leaves, etc., and prevent their clogging up the iron screens spoken of above. I crossed the breast at this time and found the water was cutting the outer face of the dam, but not as badly as I feared it would, its greatest effect was on some portions of the roadway which crossed the breast where the roadway had been widened on the lower side by the addition of a shale earth or disintegrated shale, upon which the action of the water was instantaneous, but the heavy rip-rapping on the outer face of the dam protected this wash and the water cut little gullies between each of the large stones for rip-rap. I did not stay on the dam when it was in that condition, but went on to the end of the dam and found that over its entire top it was serried by little streams where the water had broken through our little embankment and was running over the dam. I went on to the new waste-weir we had cut and found it carrying off a great volume of water and at a great velocity. I with difficulty waded it and found that it was up to my knees or 20 inches deep. I felt confident that nothing more could be done to save the dam unless we were to cut a wasteway through the dam proper at one end and allow it to cut away in but one direction, and that towards the center of the dam, but this I would not dare to do, for it meant the positive destruction of the dam, and the water at the time was almost at a stand, owing, without doubt, to the large increase of outlet by the overflow on the breast, and I hoped that it would not rise, but yet expected it to rise for it had been raining most all of the morning, and consequently we had more water to expect. I hurried to the club house to get my dinner and to note the height of the water in the lake, and found that it was a little over a stake, that from my level notes of a sewer I was constructing I knew was 7.4 feet above the normal lake level. I returned to the dam and found the water on the breast had washed away several large stones on the outer face, and had cut a hole about 10 feet wide on the outer

face and about 4 feet deep, the water running into this hole cut away the breast in the form of a step both horizontally and vertically, and this action went on widening and deepening this hole until it was worn so near to the body of the water in the lake that the pressure of the water broke through, and then the water rushed through this trough, and cut its way rapidly into the dam at each side and the bottom; and this continued until the lake was drained. I do not know the actual time it consumed in passing through the breach, but it was fully 45 minutes. It did not take long from the time that the water broke into this trough until there was a perfect torrent of water rushing through the breast, carrying everything before it, trees growing on the outer face of the dam were carried away like straws. The water rushed out so rapidly that there was a depression of at least 10 feet in the surface of the water flowing out, on a line with the inner face of the breast and sloping back to the level of the lake about 150 feet from breast, exactly similar to water flowing through a rectangular sluice-way in the side of a trough with the water level far above the bottom of the sluice-way. When the lake was drained there still remained in the bed of it a violent mountain stream 4 or 5 feet deep, with a swift current, the combination of the two streams already alluded to from the head of the lake and the many little streams from the adjacent hills, which streams were all overflowing their banks, this stream in the bed of the lake showed no signs of diminishing in volume until late in the following day, and was impassable with a boat for several days.

I need say nothing of the character of the dam, for it is open for inspection of those far more able to express an opinion than I. But there is one thing I want to impress on every one's mind, and that is, that the dam did not break, but was washed by the water passing over it from 11.30 o'clock A.M. until nearly 3 P.M. until the dam was made so thin at one point, that it could not withstand the pressure of the water behind it, and the water once rushing through this trough nothing could withstand it.

Hoping this report will be of some service to your Society, and placing myself at your disposal to answer any inquiries as to the destruction of the dam or any points that I have not developed, believe me,

Respectfully yours,

JOHN G. PARKE, JR.

AMERICUS, Ga., August 22d, 1889.

By the survey of the reservoir made for us by Mr. William H. Scriven, its area at the ordinary height of the water was 407.4 acres. At 5 feet above the ordinary height, 456.8 acres. On account of the obstruction at the head of the wasteway caused by the fish guard, and the sill on which it was placed, the ordinary height of the water in the

reservoir must have been about a foot above the average height of the floor of the wasteway.

Mr. Parke's observation of the rise of the water in the reservoir of 9 inches in an hour, combined with the area of the reservoir, enables us to compute the rate of accumulation of water in the reservoir just before it commenced running over the embankment. To find the rate at which the water was flowing into the reservoir at the same time, it is necessary to ascertain the rate at which it was flowing out through the wasteway. This is a more uncertain matter, as we have been unable to find any rule or principle of hydraulics directly applicable to the case. We have, however, attempted to approximate to it by three methods which we have worked out separately, and give in some detail in the appendix to this report. The second method, being founded more directly on experiment, we have most confidence in, and adopt.

Mr. Parke's observation of the rise of 9 inches in an hour was evidently made when the height in the reservoir averaged about $7\frac{1}{2}$ feet above the floor of the wasteway. At this height the area of the reservoir was 471.62 acres, and the accumulation was at the rate of

$\frac{471.62 \times 43\,560 \times 0.75}{60 \times 60}$	4 280	cubic feet	per second.
By our second method the discharge through the wasteway at this height was....................	3 700	"	"
Total quantity entering reservoir at 11.30 A.M....................	7 980	"	"

Earlier in the day, say at 10 A.M., the rise in the reservoir was probably more rapid, it was currently reported to have been 10 inches an hour. There is no probability that it exceeded 12 inches an hour. At that rate, when the height in the reservoir was 5 feet above the floor of the wasteway, the accumulation would have been at the rate of

	5 408	cubic feet	per second.
Discharged through wasteway by our second method................	1 800	"	"
Total quantity entering reservoir....	7 208	"	"

Just previous to the breach in the embankment the water was flowing over a large part of its length. From the statements of Mr. Parke

and others who saw it, we gather that the flow was equivalent to 100 feet in length 1 foot deep, and 300 feet in length 9 inches deep. We estimate the quantity flowing over at

	991	cubic feet	per second.
The accumulation in the reservoir we estimate to have been 6 inches in depth in the last hour, equivalent to	2 911	"	"
The discharge through the wasteway when the height in the reservoir was 8.71 feet above the floor of the wasteway by our second method was	4 780	"	"
Estimated flow into the reservoir at the commencement of the breach, say at 0.45 P.M.	8 682	"	"

These three estimates of the flow into the reservoir indicate that it increased up to the time of the breach, and no doubt continued to do so for some time longer. Two-thirds of an inch of rain per hour on the water-shed is equivalent to 20 909 cubic feet per second. The rate of flow caused by a given rain-fall from any water-shed, in the absence of direct measurement is always difficult to estimate; it depends on several conditions:

First.—The length of time during which the rain-fall at any given rate has continued, it being obvious, that the longer it continues the nearer the rate of flow from the water-shed will approximate to the rate of the rain-fall.

Second.—The extent of the water-shed; the larger it is, the longer will be the average time required to reach the reservoir.

Third.—The general inclination of the surface, on which depends the rapidity of the flow from the water-shed.

Fourth.—The character of the water-shed as to its capacity to hold back the water; the favorable conditions for discharging rapidly being rocky and cleared surfaces; and for discharging slowly; ponds, lakes, swamps, large level tracts and woodland.

In this case the extreme length of flow does not exceed 10 miles, and the average length probably is less than 4 miles. The inclination of the

surface and the character of the water-shed appear to be generally favorable to a large discharge, and taken in connection with the continuance for several hours of heavy rain-fall, it would appear that a maximum rate of flow into the reservoir of one-half the rate of the rain-fall would not be too large an estimate. This would give a flow into the reservoir for a rain-fall of two-thirds of an inch an hour of $\frac{20\,909}{2} = 10\,454$ cubic feet per second.

This considerably exceeds the estimates of the flow just previous to the breach, but is not inconsistent with it as a maximum. Mr. Parke's observation just before the commencement of the breach, that "it had been raining most all of the morning and consequently we had more water to expect," and on the strong current in the bed of the lake immediately after the breach, and its long continuance subsequently, indicate a possible increase of the flow to this extent. The accompanying diagram, Plate LII, of the estimated flow into the reservoir during the storm, indicates that if the maximum flow was 10 000 cubic feet per second, it would occur at about 4 P.M. of May 31st, or about three hours after the breach; but we have no proof that it would not have been later and the maximum flow correspondingly larger.

The breach in the embankment is about 420 feet wide at the top and 50 to 200 feet wide at bottom, the amount of earth and stone in the same being about 90 000 cubic yards, Plate L. All the material put in in 1880 and 1881, to repair the breach of 1862, appears to have been washed out, together with part of the old embankment made in 1851 and 1852. Some parts of this old work are exposed by the flood, and indicate that it offered great resistance to washing and that it was originally selected and put in with the requisite care to make a sound embankment. The original construction of the embankment as indicated by the plan and specification, and more particularly the mode of repairing the breach, may be objected to as not being according to the best practice; nevertheless, the failure of the dam cannot be attributed to any defect in its construction. The failure was due to the flow of water over the top of the earthen embankment, caused by the insufficiency of the wasteway to discharge the flood water.

Recapitulating, briefly, what we have ascertained relating to the wasteway: In 1835, Mr. Sylvester Welsh, then principal engineer of the Canal Commissioners, recommended the site for the dam and proposed a wasteway at one or both ends of the dam of sufficient size to discharge the water waste during freshets and stated that "no water would be permitted to pass over the dam." In a report dated November 1st, 1839, Mr. William E. Morris, then principal engineer, recommends for the dam an embankment of stone and earth having a wasteway at one or both ends of the dam for the passage of flood water, and in the specification prepared by him for the execution of the work which was let out by contract in 1839, it was provided that the wasteway excavated in the hill at one or both ends of the dam, shall have an aggregate width of channel of not less than 150 feet. Little progress was made on the work until 1846, when Mr. Morris, although no longer in office as the principal engineer of the Canal Commissioners, was called upon by them for plans and specifications for a new letting. They were the same as those prepared by him in 1839, except that a frame tower was substituted for one of masonry. Nothing appears to have been done until 1851, when the work was proceeded with and completed in 1853. The wasteway was, however, not constructed according to Mr. Morris' specification; instead of having an aggregate width of 150 feet, its effective width was less than 70 feet. The width at its entrance, where it is the widest, is about 120 feet, and at the outfall, 176 feet from the reservoir, it is about 69 feet, which must be taken as its effective width. The horizontal bed of the wasteway from the reservoir to the outfull, a distance of about 176 feet, sensibly diminished its efficiency, over what it would have been if the outfall had been at the entrance of the wasteway.

Assuming the maximum flow into the reservoir to have been 10 000 cubic feet per second, and the wasteway to have been constructed according to Mr. Morris' specification, with a free outfall at the entrance, we estimate that the corresponding height in the reservoir would have been 7.80 feet above the crest of the wasteway, or 2.20 feet below the top of the embankment as specified by him. With the five lines of 24-inch sluice pipes discharging to their full capacity at the same time, under 70 feet head, as no doubt was originally designed to be done in an emergency, we estimate the height in the reservoir would be about 7.52 feet or 2.48 feet below the top of the embankment.

The height of the embankment as constructed in 1851–53, was accord-

ing to Mr. Morris' plan and specification. According to our method of estimating the flow through the wasteway, the discharge through it, when free from obstruction, and the water in the reservoir just up to the top of the embankment as originally constructed, and the sluice pipes discharging to their full capacity, would be 6 923 cubic feet per second. For the wasteway and sluices, as constructed in 1851–53, to discharge 7 980 cubic feet per second, which we have estimated above to have been the flow into the reservoir at the time of the breach, the height in the reservoir under the same conditions would be 10.95 feet above the floor of the wasteway or about 1 foot above the top of the embankment as originally constructed. For the discharge of the maximum quantity entering the reservoir, which we think was not less than 10 000 cubic feet per second, under the same conditions, would require a depth of about 12.63 feet.

The Hunting and Fishing Club, in repairing the breach of 1862, took out the five sluices in the dam, lowered the embankment about 2 feet, and subsequently, partially obstructed the wasteway by gratings, etc., to prevent the escape of fish. These changes materially diminished the security of the dam, by exposing the embankment to overflow, and consequent destruction, by floods of less magnitude than could have been borne with safety if the original construction of 1851–53 had been adhered to; but in our opinion they cannot be deemed to be the cause of the late disaster, as we find that the embankment would have been overflowed and the breach formed if the changes had not been made. It occurred a little earlier in the day on account of the changes, but we think the result would have been equally disastrous, and possibly even more so, as the volume of water impounded was less, and the greater width of the top of the embankment after the change and its consolidation by its use as a road for several years, must have increased its resistance to the formation of a breach and required more time.

In concluding, we must state that, while our deductions are based on the results of observations of rain-fall and of flow which are necessarily approximate, we feel satisfied that they are not far from the truth. There can be no question that such a rain-fall had not taken place since the construction of the dam. But the surface of the water-shed is quite steep, and the consequent rapid discharge of a large percentage of the rain-fall into the reservoir would require a very large outlet to prevent a dangerous accumulation. The spillway, however, had not a sufficient

discharging capacity; contrary to the original specifications of Mr. W. E. Morris, requiring a width of overflow of 150 feet and a depth of 10 feet below crest, which would have been a sufficient size for the flood in the present case, it had only an effective width of 70 feet, and a depth of about 8 feet; the accumulated water rose to such a height as to overflow the crest of the dam and caused it to collapse by washing it down from the top.

The dam itself, or the parts of it which were left standing, showed undoubtedly that it was well and thoroughly built, and that it would have successfully resisted the pressure of the water. The exposed sides of the breaks show distinctly that the compact layers of which the structure was formed were not obliterated by the wearing action of the flood, and they stand conspicuous witnesses of the value of an earth embankment when well built of good materials, to impound large bodies of water.

There are to-day in existence many such dams which are not better, nor even as well provided with wasting channels as was the Conemaugh Dam, and which would be destroyed if placed under similar conditions. The fate of the latter shows that, however remote the chance of an excessive flood may be, the only consistent policy, when human lives, or even when large interests are at stake, is to provide wasting channels of sufficient proportion and to build the embankment of ample height.

Respectfully submitted,

James B. Francis,
W. E. Worthen,
M. J. Becker,
A. Fteley.

January 15th, 1890.

APPENDIX.

Estimates of the Discharge through the Wasteway and Sluice Pipes.

The wasteway of the South Fork Reservoir in plan, is about 120 feet wide at the reservoir, narrowing to about 67 feet at 176 feet from the reservoir, the axis being nearly a quadrant of a circle of about 100 feet radius. In profile the average level of the bed of the wasteway for 176 feet

from the reservoir is nearly horizontal (see Fig. 4, Plate XVI). The noted elevations varying from 0.14 feet above the average to 0.08 feet below. In the next 50 feet a descent of a foot is noted. This is evidently amply sufficient to maintain the velocity acquired by the water at the end of the horizontal part of the said bed, and consequently the bed beyond this point offers no resistance to the flow over the horizontal part, and this point is therefore taken to be the outfall of the wasteway.

To compute the discharge through a wasteway of this peculiar form we have not found any formula or established principle in hydraulics that will enable it to be done directly. The three following methods of approximating to it occur to us, which we have worked out separately, viz.:

First Method.

In the first place it is assumed that the bed offers no resistance and that the flow adjusts itself so that the discharge is a maximum, which we find is when the descent of the surface of the water from the reservoir to the outfall, is one-third and the depth of the stream at the outfall two-thirds of the height of the water in the reservoir above the bottom of the wasteway. This is based on the idea that the velocity at the outfall is that due to the descent of the surface $\frac{D}{2}$, and that this applies to the whole depth of the stream D. The assumed height in the reservoir being $1.5D$.

In working this out it is assumed that the velocity through the section of the stream at the outfall is that due to the head $\frac{D}{2}$ with a coefficient of contraction of 0.9, and the discharge Q is the product of this velocity into the area of the section at the outfall.

To find the real height in the reservoir, there must be added to the assumed height $1.5D$ a head equivalent to the resistance of the bed when the quantity, computed as above, is flowing. This is done by means of the Chezy formula: $V = 75\sqrt{RI,}$ from which we deduce for the channel 176 feet in length, putting $H =$ the required head and $R =$ the hydraulic mean depth.

$$H = \frac{0.0313\ V^2}{R}.$$

To allow for the variations in the width and depth of the wasteway (see Sec. 1, Plate XLI) we determine V^2 and R for the several sections of the channel and take their mean values. As H is a factor of V and R at each section, we arrive at the result by successive approximations. The height in the reservoir corresponding to the computed discharge for the depth R at the outfall is $1.5D + H$.

The above computation is for the case where there is no obstruction at the entrance of the wasteway. When obstructed by the fish-guard

and posts supporting the bridge, as it was when the dam gave way, an addition to the height must be made equivalent to the obstruction. The top of the fish-guard is understood to be 2.14 feet above the bottom of the wasteway, and the area of the obstruction to this height, including the posts supporting the bridge to the same height, we find to be 120.43 square feet. Above the top of the fish-guard the posts supporting the bridge were the only obstructions, and for each foot in height they add 7.9 square feet, and for any height H^1 in the reservoir greater than 2.14 feet, the area of the obstruction is $120.43 + 7.9\ (H^1 - 2.14)$ square feet.

To find the additional height due to the obstruction, the area obstructed is subtracted from the total area of the section at the entrance of the wasteway, the difference being the area through which the water enters the wasteway, and the velocity found for each area and also the heads due these velocities. In computing them a co-efficient of contraction 0.9 is used for the unobstructed section and from 0.6 to 0.8, depending on the height in the reservoir for the obstructed section. The difference in these heads $h^1 - h$ is the addition to be made to the height in the reservoir corresponding to the discharge, the total height being $1.5D + \mathrm{H} + h^1 - h$.

Second Method.

The discharge is considered to be over a weir or dam with a wide crest. Experiments were made at Lowell, in 1852, and are recorded in Lowell Hydraulic Experiments, on such a dam in which the crest was about three feet wide, in this case it will be 176 feet wide, and to give the height in the reservoir it is necessary to add to the depth at the outfall a head equivalent to the resistance to the flow over this width of crest. The formula given with the record of the experiments is determined from the discharge with depths from 0.59 feet to 1.63 feet. As being better adapted to the present purpose, the formula is determined anew from the experiments in which the depths are 1.32 feet and 1.63 feet, from which we find $Q = 3.056\ L\ H^{\frac{3}{2}}$ in which L is the effective length of the weir, and is obtained by dividing the area of the section at the outfall by the depth H. The additional height in the reservoir equivalent to the resistance to the flow over the wide crest is determined in the same manner as the head equivalent to the resistance of the bed in the first method, and the additional head when the entrance to the wasteway is obstructed by the fish-guard, etc., is determined in the same manner as in the first method.

Third Method.

It is assumed that the discharge is through an orifice converging laterally, but of a uniform height, the same as the depth at the outfall, the orifice being of the same area as the section of the stream at that

point, with a velocity due to a fall equal to one-half the depth at that point, computed with a co-efficient of contraction of 0.9, and to find the corresponding heights in the reservoir, the heads, equivalent to the resistance of the bed and to the obstruction, if any, at the entrance to the wasteway, are added to the depth at the outfall. These heads are computed in the same manner as in the first and second methods.

The quantities discharged and the corresponding heights in the reservoir by the three methods are plotted on the diagram, Plate LI, and curves drawn through the several determined points.

The quantity of water discharged by the five lines of 24-inch sluice pipes shown in elevation and plan in Plate LI, computed by the formula deduced from Darcy's experiments on the flow through old cast-iron pipes, given in Paper No. 37, in the *Transactions* of the Society for 1872. The head due the initial velocity is computed by the usual formula, using 0.82 as the co-efficient of contraction. The length of the pipes by the original plan is about 100 feet, and the effective head, when the water in the reservoir is 7 feet above the bottom of the wasteway and both pipes and wasteway are discharging to their full capacity, is taken at 70 feet. With these data, the discharge, of the five pipes, is found to be 543 cubic feet per second.

LIST OF PLATES.

DISCUSSION.

P. F. BRENDLINGER, M. Am. Soc. C. E.—Mr. President, I would like to discuss the report, and make a few remarks about the plans submitted by the committee. I took some levels and measurements at the dam about a year ago and I find quite a discrepancy between my measurements and those given here by the committee. I presume the committee made their own measurements, or employed competent men to do it.

If I mistake not, Mr. Park in his report said, when he got down to the dam there were about 7 feet of water in the spillway and the water then was not running over the dam. Let us stop right here and take up the plate showing the "longitudinal section through center line of dam" and "top view of dam." I am extremely sorry that the crest of the dam is not shown throughout, or, in other words, that the committee has not deemed the matter of sufficient importance or wise, to ascertain or locate the crest line of the dam as it existed before the flood or destruction of the dam. To my mind this is an extremely important link to establish, nor do I think it such a difficult matter to locate, and prove that the crest was no higher, while it may have been lower.

I took very careful levels and measurements about a year after the destruction of the dam, and I find the levels differ from those shown here in the plate about $\frac{1}{10}$ of a foot per hundred, uniformly, in other words, I make the heights from the top of the spillway to the crest of the dam $\frac{1}{10}$ of a foot less than the committee—in all other respects my measurements agree with those submitted by the committee. As my observations were made almost a year after the committee made theirs, the difference in levels may be caused by rains washing the crest of the dam away to a certain extent. Or different points on the top of the sill in the spillway may have been used as the proper place for the elevation of the same. I find there is a difference of about 18 inches between the heights of the bridge at the ends. Whether it is in the bridge or sill, or partly in both, I do not know. I know the height of

the bridge varies. I took the height of the top of sill at a point on the dam side 26 feet from the dam end of the bridge. It is not likely that the committee's elevation of top of sill in spillway was taken at this identical spot, hence the difference of four-tenths may be accounted for in this way. I will therefore discard my levels for the present and present a "longitudinal section through center line of dam," made by myself from the levels and measurements shown on the plan above mentioned (see Plate LIII). This is in fact a reproduction of the committee's plan. I have added to it the "Missing Link Projected" in a heavy dotted line—from point "*A*" to "*B*." I find Station 9 + 50 to have an elevation of 1610.87, and Station 8 + 50 to have an elevation of 1610.07, there is, therefore, a fall or grade of minus eight-tenths of a foot in 100 feet, and at Station 9 + 100 this same grade makes the elevation 1610.47, which is correct. Producing this grade, we have an elevation at Station 8 of 1609.67 and at 7 + 75, the edge of the break, 1609.47 which is just one-tenth of a foot higher than the elevation of the ground as given, but as the rains may have washed the ground slightly we may therefore consider the grade on the "right side" of the crest of the dam as established at — 0.8 per 100 feet from Station 9 + 50. On the left side, unfortunately, we have but a short part of the dam left of a uniform grade, but it is sufficint to determine the grade. The difference between Stations 3 and 3 + 50 is six-tenths of a foot or at the rate of 1.2 feet per 100. Continuing the two grades, the — 1.2 on the left side and the — 0.8 on the right side, the point of intersection is found at Station 5 + 78 at an elevation of 1607.90 or 4.5 feet above the spillway.

The point or points of change from a down grade to a level or rising grade can only occur somewhere between the points *AF* and *BF*, hence I will give the position of the center at *F*, all the benefit in height I can, by fixing the point of curvature at *B*, and projecting a curve whose tangential distances are *BF* and its equal *FK*.

The curve or change may have commenced nearer *F* than *B* or *K*, which would have reduced the height of the dam, but it is impossible to have commenced either between *BE* or *AS*, as the levels plainly show. It is quite evident that the curve or missing link could not have been reversed, as that would not only have been unsightly to the members of the South Fork Fishing Club, but the road would have been hard to use, and besides the water would not have gone over it. It is also highly improbable that *B* was the lowest point and the grade changed to a rising grade from there to *A*, for that would have been noticed by everybody, if not while walking on the road, then from a side view. I regard it as exceedingly strange that there was or is no map of the South Fork Fishing Club's property showing a longitudinal section of this dam before it was destroyed. This club was composed of the best families of Pittsburgh, mostly men of extensive wealth and

business experience, who do nothing by halves. They had a civil engineer in their employ whose report to the committee has highly pleased and gratified me and no doubt all the members, and I must say it occurs to me as being very singular that there should be no such section as above mentioned in existence, and I am perfectly satisfied that the dam was not any higher than the "Missing Link Projected" shows, but may have been lower. I will now give a table of actual elevations as taken, and those measure on the map:

Station.	Elevation.	Ht. above Spillway.	
1................	1610.44	7.04 feet.	
1 + 36 (Ditch).....	1606.84	3.44 "	
1 + 50............	1610.04	6.64 "	
2................	1610.84	7.44 "	
2 + 50............	1611.14	7.14 "	
3................	1611.24	7.84 "	
3 + 50............	1610.64	7.24 "	
4................	1610.00	6.60 "	
4 + 50............	1609.50	6.20 "	
5................	1609.10	5.70 "	
5 + 50............	1608.80	5.40 "	Projected
6................	1608.70	5.30 "	Missing Link.
6 + 50............	1608.80	5.40 "	
7................	1608.90	5.50 "	
7 + 50............	1609.20	5.80 "	
7 + 75............	1609.37	5.97 "	
8................	——	——	
8 + 50............	1610.07	6.57 "	
9................	1610.47	7.07 "	
9 + 50............	1610.87	7.47 "	
10................	1611 87	8.47 "	

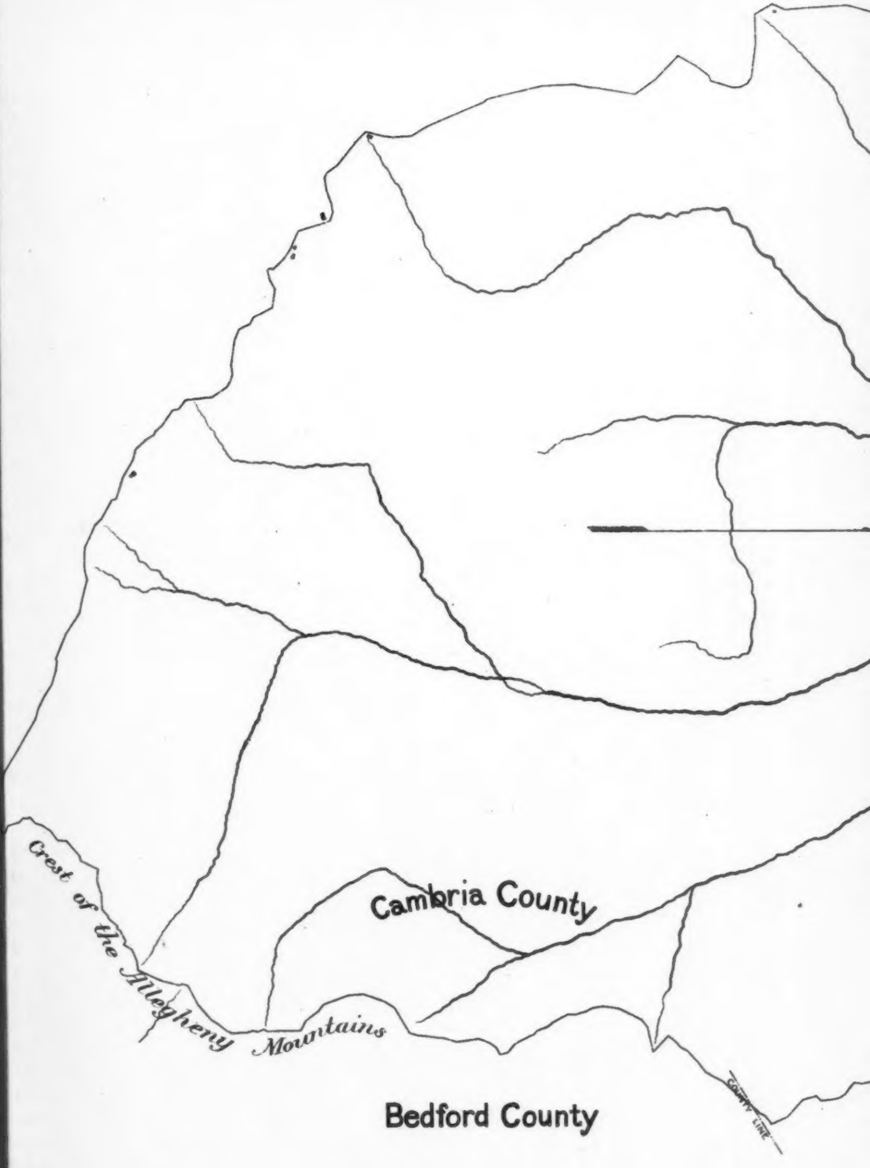
It is a great pity that the elevation of Station 8 was not taken or put on the plat for it would assist very materially in properly determining the height of the dam. Please observe that the lowest point on the "Missing Link Projected" is at Station 6, and is 5.3 feet above the spillway, and is only 0.67 feet, or 8 inches lower than the edge of the break at *B*. I do not believe any one will, for a moment, doubt that these two opposing down grades towards a common intersection point were connected with an easy inverted vertical curve whose lowest elevation was at least 8 inches below *B*. Various parties have told me the water was deepest in the middle, and parties who went across the dam when the water commenced going over, found the water nearly to the top of their gum-boots in the middle, and only in the middle.

Coming back now to the first part of my remarks, viz.: that when Mr. Park, the engineer for the South Fork Fishing Club, came to the dam he found about 7 feet of water in the spillway, and the water then was not running over the dam. There is a big mistake here somewhere in Mr. Park's statement or in my understanding, or in the committee's levels and my deduced levels—for if there had been 7 feet of water in the spillway there would have been water over the dam to the extent of 1.03 feet at *B*, and 1.7 feet at Station 6.

The depth of the ditch seems to me to have been erroneously take The elevation of the bottom is given as 1606.84 or 3.44 feet above th spillway—this makes a depth of 3.2 feet in red shale rock, which seem to me to be impossible to excavate in such a short time, besides, this i 2 feet lower than my lowest projected elevation of the dam and such quantity of water would have gone through this ditch as to tear up th ground below quite lively. I did not see any evidences of this and i fact would not have observed any ditch or depression if I had not rea of it and consequently looked for it, I found a very slight depression be low the surrounding general level.

From the report of the committee I understand they attribute th destruction of the dam to the inadequate spillway at *G H*, a poi about 175 feet below the bridge, crossing the spillway. I find, fro the plat presented by the committee, this, its narrowest cross-sectio to have a width of 64 feet at a 4-foot stage of water. I do not kno how far the committee went into the examination of the obstructio caused by the bridge and the trestles with the fish-screens under i but for the information of the members, I present an elevation of th bridge with its barnacles attached as I saw it. As I said before, there i a difference of 18 inches or more between the ends in height of th bridge above the sill, but whether the sill or bridge is level I do n know. The average height from the top of sill to the bottom of th floor is 10 feet, so I drew my bridge parallel with the sill for convenienc All parts and spans showing figured dimensions are plotted to scale, th rest are drawn from a sketch made on the ground. There are, or were year ago, still three rod screens in position, fastened to the posts, th screens are composed of ½-inch round bars passed through and held i place by top and bottom pieces of 2½ inches by ½-inch iron fastened in two vertical end pieces of the same size. The rods are about 1¼ inch center to center, leaving ¾-inch space between them, this reduces th space, to the height of the bars alone, 40 per cent.

We have at this point a length of sill of 99 feet — reduce this b fourteen posts 6 inches wide and one 8 inches = 7 feet 8 inches or feet 4 inches in all, and 40 per cent. = 36 feet 6 inches, we have a balan of 91 feet 4 inches of this 36 feet 6 inches and leaves 54 feet 10 inches. The is an additional screen which I show at the right side—it is compose of wire ⅜-inch mesh, 3 feet 7 inches high, fastened to a piece of timb 8 × 8 inches, which slides up and down as the water rises and falls b means of an eye at the end, on a rod fastened to the end-post. This pie of timber is almost submerged by its own weight, and with the wi screen attached I doubt whether there is much of it above the surface the water. The wire screen alone takes up about 25 or 30 per cent. of th area, while the solid timber takes up 8 inches in depth out of 3 fe 7 inches, or 20 per cent. of the total, making a reduction of the avai able area of 40 to 50 per cent., or reducing the width from 99 to abo


Crest of the Allegheny Mountains
Cambria County
Bedford County
COUNTY LINE

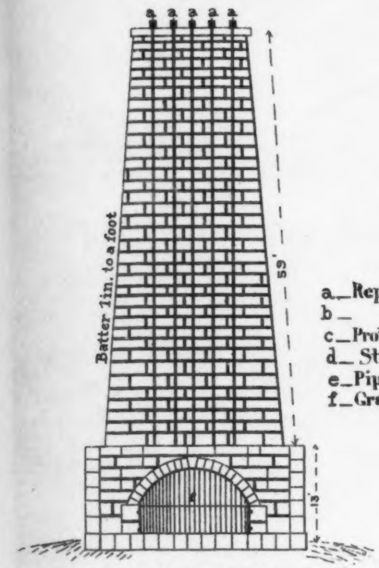

PLA
TRANS.AM.S
VOL.X
RE
SOUTH
Western or South Fork Res.
407.4 Acres.
E.J.Unger
J. Borabaugh
1mile to Wilmore
¾ mile to Wilmore
15miles to Johnstown
19miles to Hollidays
Road to Portage 2 miles
AREA OF WATER SHED OF SOUTH FORK OR WESTERN RESERVOIR
48.6 SQ. MILES.
COUNTY LINE

PLATE XLVII
ANS. AM. SOC. CIV. ENG'RS.
VOL. XXIV. No 477
REPORT ON
SOUTH FORK DAM.

DAMA

RES

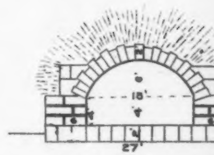
Wi



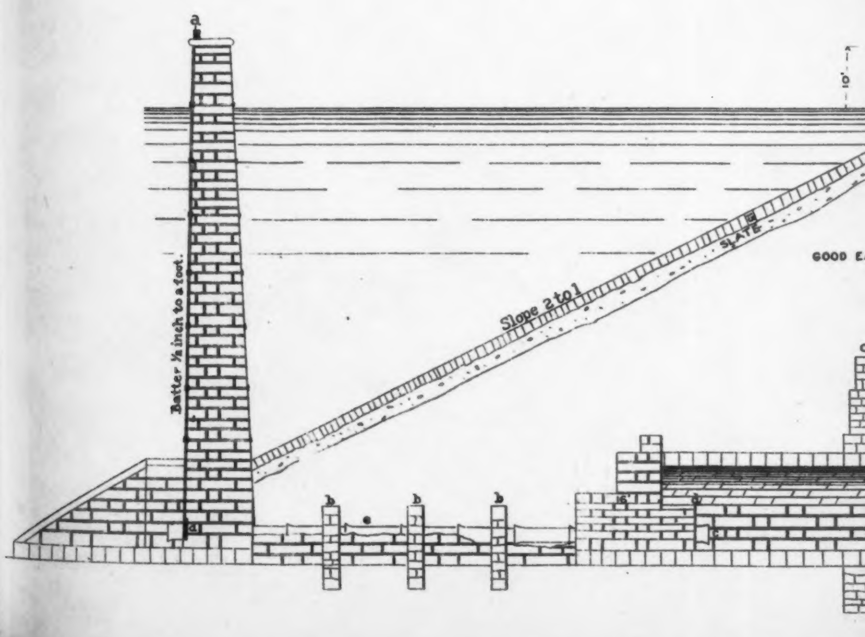
—Reference—

a._Represents valve rods.
b_ " wall to support pipes.
c_Protection wall.
d_ Stop cocks.
e_Pipes.
f_Grate.

VIEW OF UPPER END OF CULVERT



SECTION THROUGH C



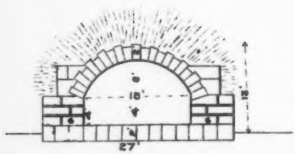
DAM AND SLUIC
for
RESERVOIRS.

BY

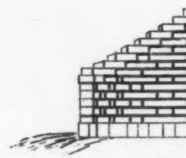
Wm E. Morris, Civil Engineer.

1839

SCALE 16 FT TO AN INCH



SECTION THROUGH CENTRE



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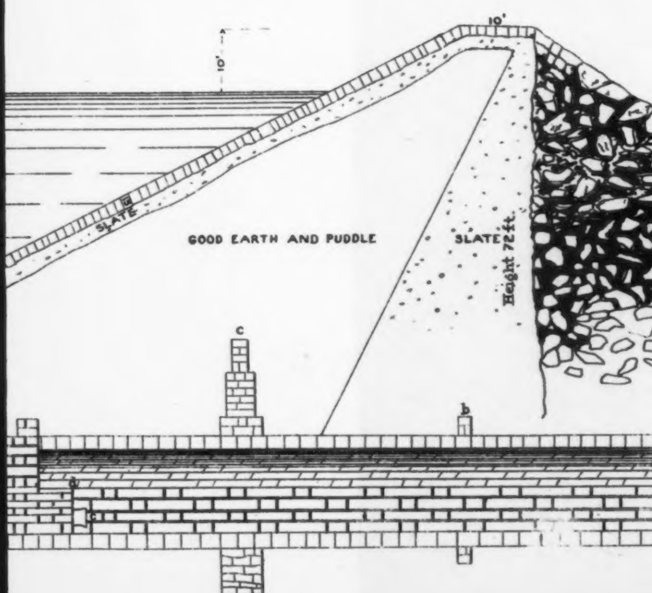
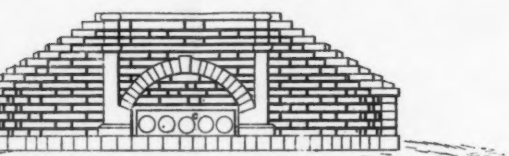


CE

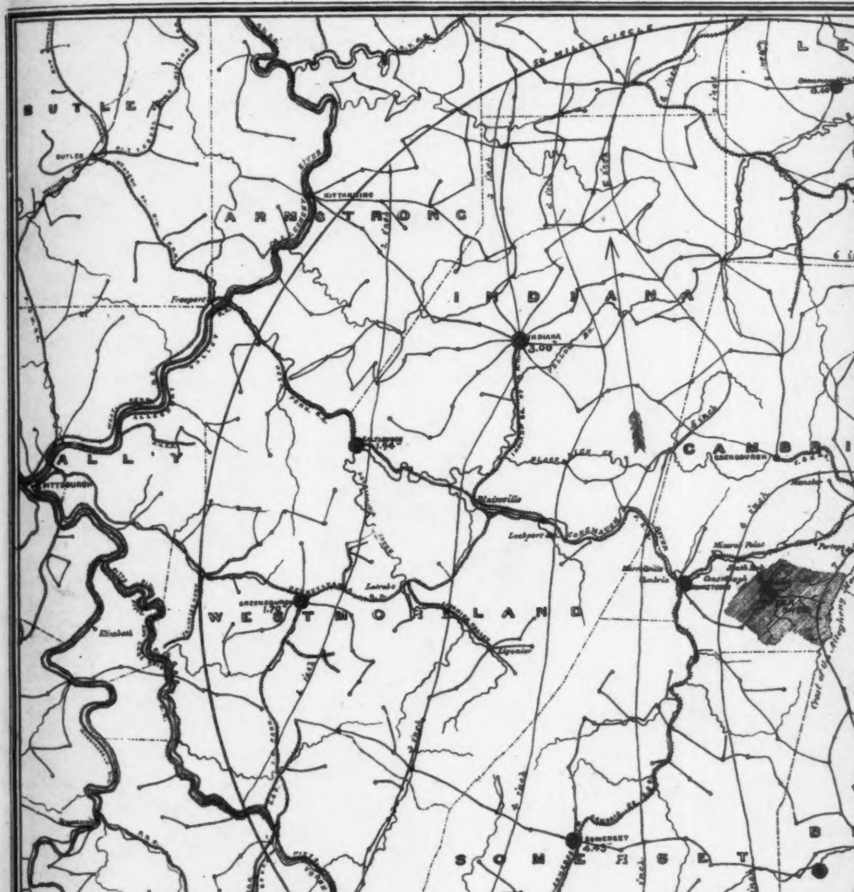
S.

PLATE XLVIII
TRANS. AM. SOC. CIV. ENG'RS.
VOL. XXIV. Nº 477
REPORT ON
SOUTH FORK DAM.

"Copy of a Plan on file in the Department of Internal Affairs of Pennsylvania."



VIEW OF LOWER END OF CULVERT



MAP SHOWING
RAINFALL WITHIN 50 MILES OF CONEMAUGH
ON MAY 30-31-1889
Scale 6 Miles to 1 inch

PLATE XLIX
TRANS. AM. SOC. CIV. ENG'RS.
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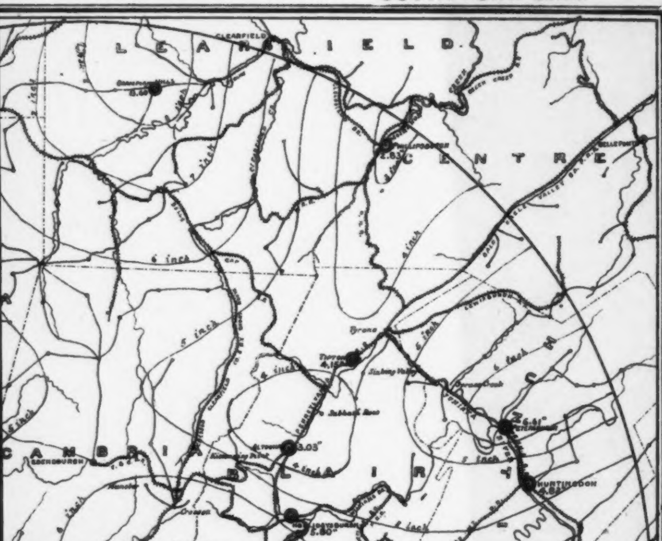
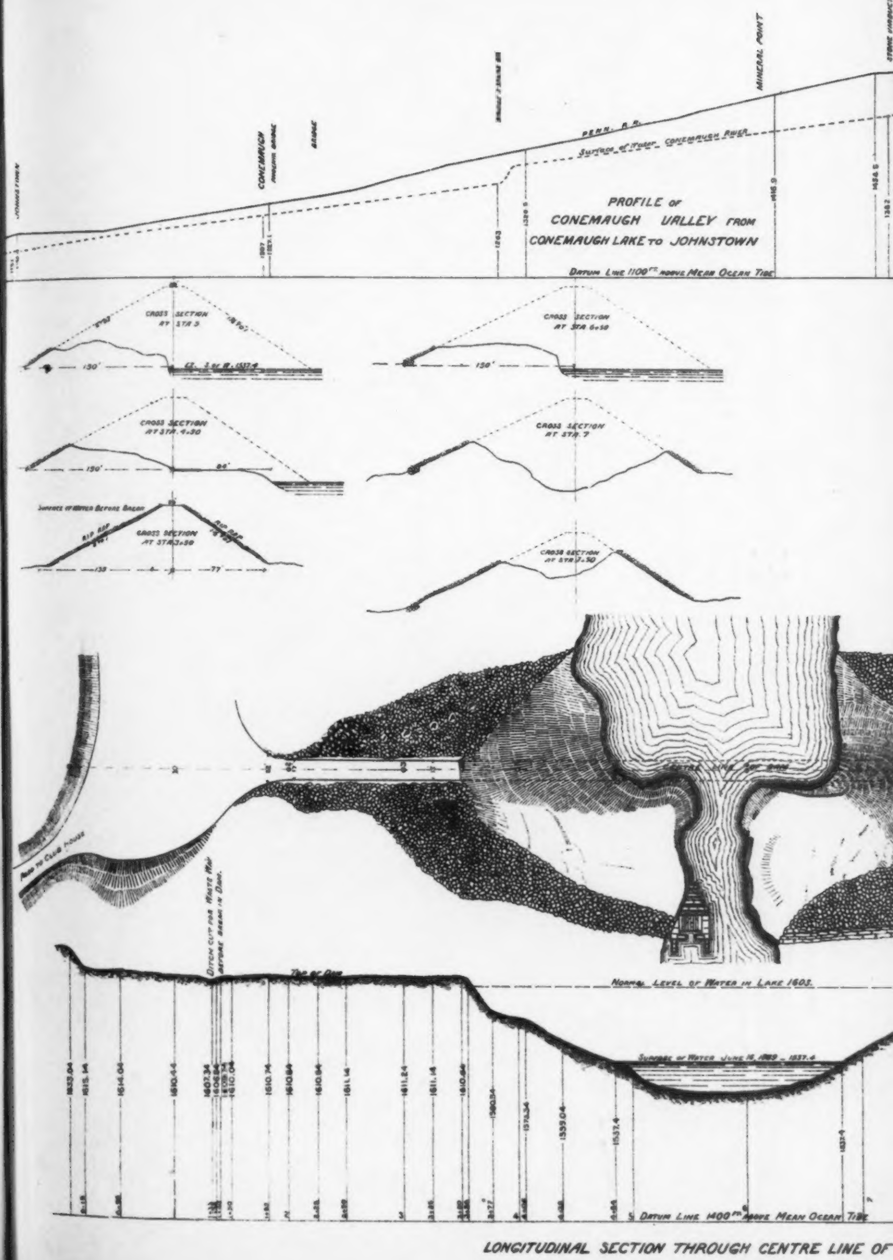
REPORT ON
SOUTH FORK DAM.



WING
CONEMAUGH LAKE PA.
31 - 1889
Note - Of this Rainfall 90 per ct fell on the 31st



LONGITUDINAL SECTION THROUGH CENTRE LINE OF

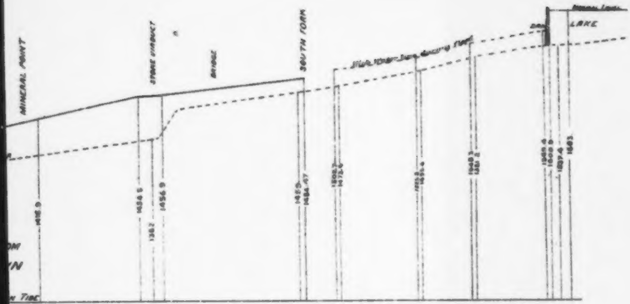
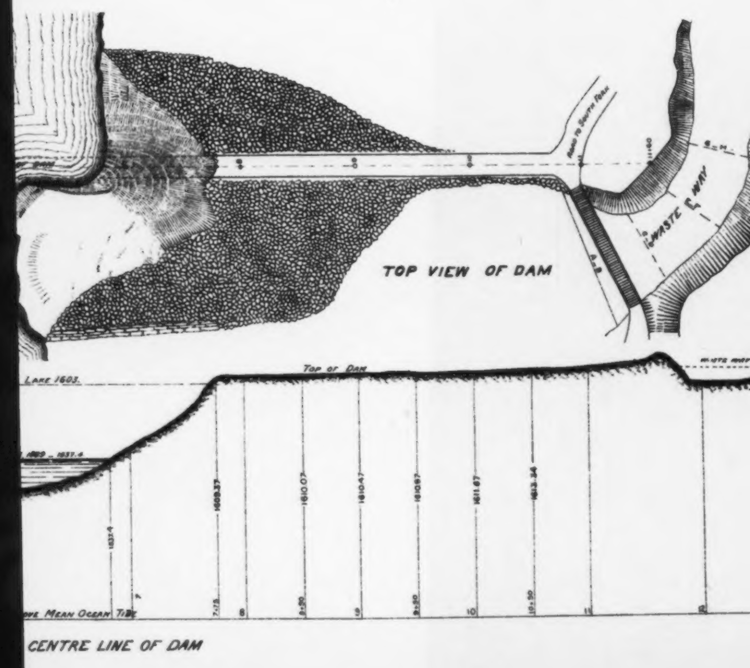
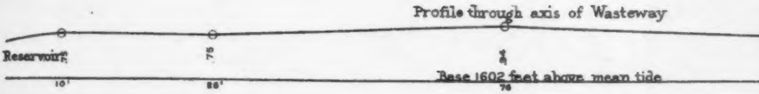


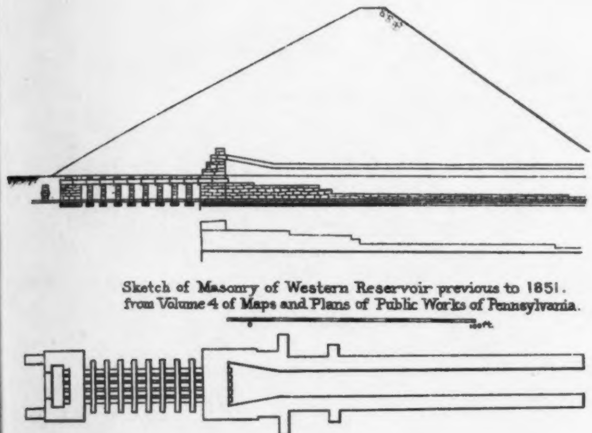
PLATE L
TRANS. AM. SOC. CIV. ENG'RS.
VOL. XXIV. Nº 477
REPORT ON
SOUTH FORK DAM.



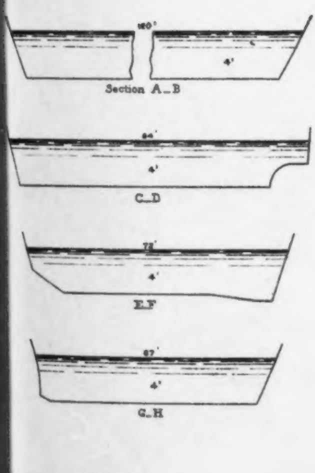
NG'RS.
7

AM.

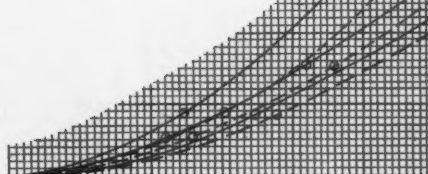




Sketch of Masonry of Western Reservoir previous to 1851.
from Volume 4 of Maps and Plans of Public Works of Pennsylvania.



The broken lines represent the discharge when the entrance of the Wasteway is obstructed by the Fishguard and supports of the Bridge. The full lines the discharge when unobstructed.



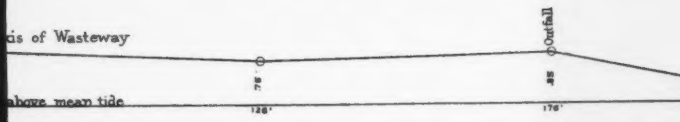
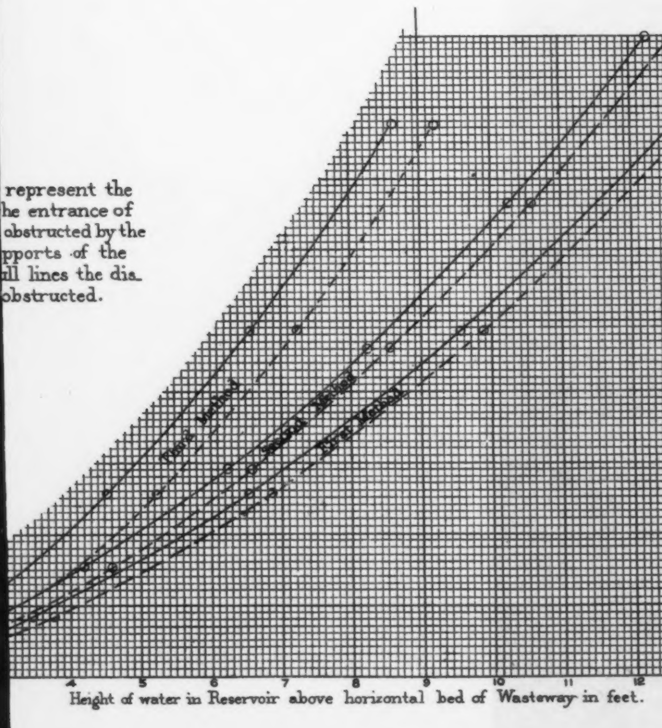


PLATE LI
TRANS. AM. SOC. CIV. ENG'RS
VOL. XXIV. Nº 477
REPORT ON
SOUTH FORK DAM.



represent the
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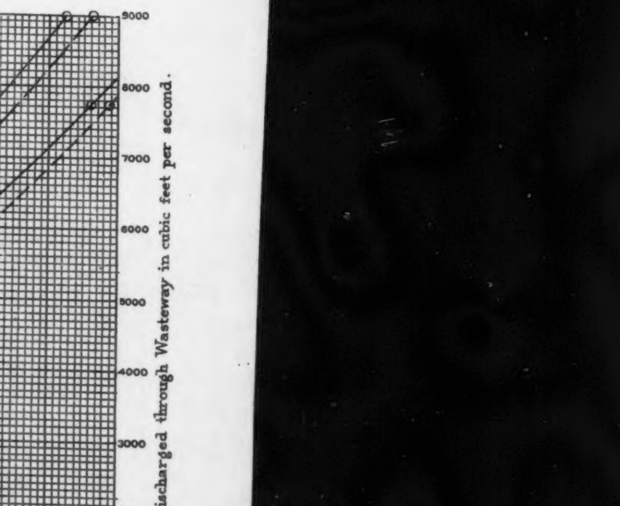
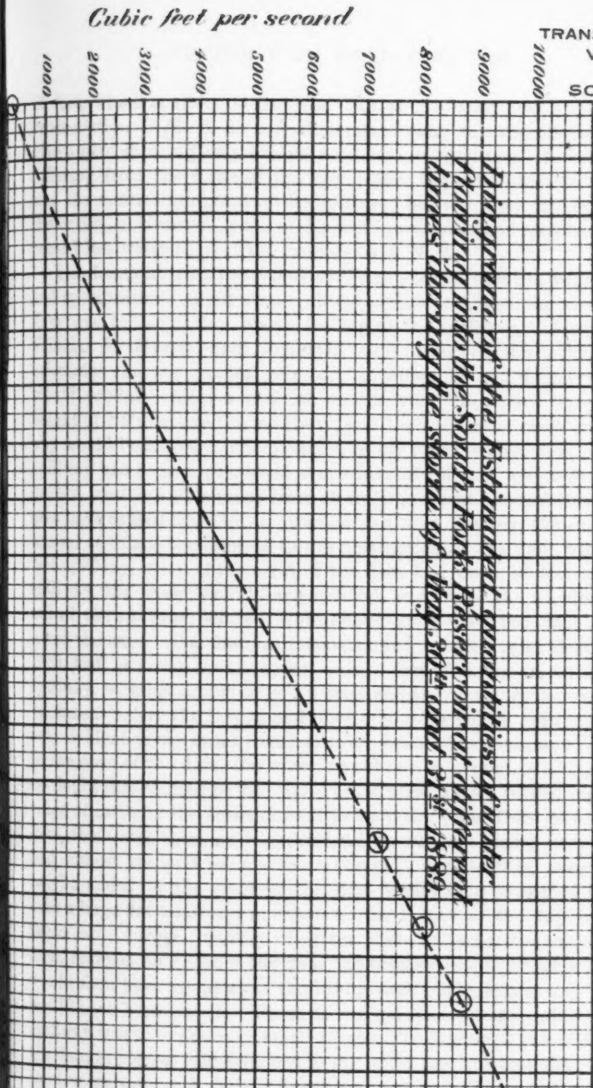
Height of water in Reservoir above horizontal bed of Wasteway in feet.

L.I
CIV. ENG'RS.
9 477
ON
K DAM.




Cubic feet per second
1000
2000
3000
4000
5000
6000
7000
8000
9000
10000
Diagram of the Estimated quantities of water
flowing into the South Fork Reservoir at different
times during the storm of May 30th and 31st 1889.

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50 feet, which is about 14 feet less than at the narrowest unobstructed cross-section *G H*. I do not understand why the committee did not give a plan showing this bridge with its obstructions. I cannot help differing from the views of the committee that the cross-section *G H* was the controlling point of the spillway, but say that the spillway at the bridge, obstructed by the bridge with its trestles and screens attached, passed less water than at the cross-section *G H*.

I understand from the report of the committee that the original plans and specifications called for a spillway of 150 feet, and the crest of the dam was to be 10 feet above the top of the spillway, Also that Mr. William E. Morris, civil engineer, who designed the original dam, calculated that the greatest rain-fall, of which he could find a record, would produce, from a rush of waters from the entire water-shed, a height of 7 feet above the spillway, and thus 10 feet of crest above spillway would be ample. The question in my mind is now, supposing this spillway had been constructed of a width of 150 feet and reduced by a bridge, trestles and screens from 40 to 50 per cent., would this reduced area have caused the dam to overflow and thus destroy the dam? My answer to this is the same as to the destruction of the dam with the 99 feet spillway, viz.: if the crest of the dam had been 10 feet above the spillway, the water would have crushed, and swept the wooden bridge out of the spillway before it would have gone over the dam and thus saved the dam. The dam, as reconstructed in 1880, and destroyed in 1889, had, as far as we know, not quite 6 feet of embankment above the spillway, without doubt only 5.3 feet, and the water went over the embankment before the bridge and obstructions could be torn out. It is very evident to my mind that had there been a few feet more in height to the crest of the dam, the water would not have gone over the dam, as no doubt this seemed to be Mr. Park's idea, as he turned up a little ridge of earth on the embankment to keep the water from going over the dam, the water then, as I understand it, was not rising very fast, but had almost reached its maximum height, besides this, every foot in height added to the crest of the dam would tend to spread the water over a large flat territory, in other words, 1 foot at the top frequently backs up a larger quantity of water than 10 feet at the middle of the dam, besides, no doubt if the dam had been several feet higher the water would soon have swept away the bridge and obstructions and the dam would have been saved. There is no question in my mind that the spillway was too narrow or small to speedily discharge the waters from the dam in a great rain-fall, but to my mind the greatest danger lay in the low embankment, only 5.3 feet, or at most 5.97 feet above the spillway and the obstructions in the spillway.

As to the material in the dam, I do not propose to repeat what I have already said at previous meetings of the Society how that dam was built; I only wish to say one word, that is, there is certainly a mistake some-

where, either in my recollection or in the report of the engineer that was read this morning, viz.: that the material was rolled when the dam was rebuilt in 1880. As I saw it, it was dumped in place out of carts, the same as material is put in railroad embankments. Whether it was rolled subsequently to that, I do not know.

In regard to the little embankment that was thrown up on the crest of the dam, I found very little evidence a year afterwards that there had been more than probably six inches altogether of a ridge thrown up.

Mr. Wilkins.—I think Mr. Brendlinger has misunderstood Mr. Park's statement in regard to the 7 feet, etc.; it was above the dam, nearly a mile, and I think very likely the difference of four-tenths would be in the hydraulic grade between the cottages and the dam.

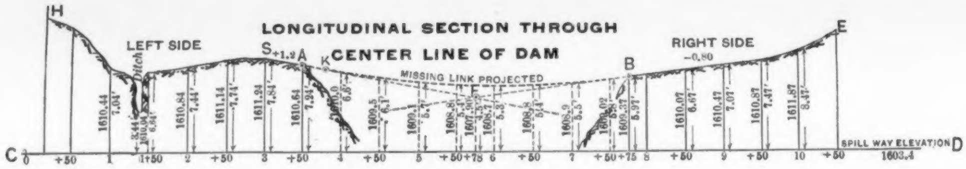
Robert Moore, M. Am. Soc. C. E.—I presume that the basis of the levels was the same in both cases, was it not?

Mr. Brendlinger.—Yes; they took the top of the sill to start; I started on top of the sill with my levels, so that I would have a comparison from the same basis.

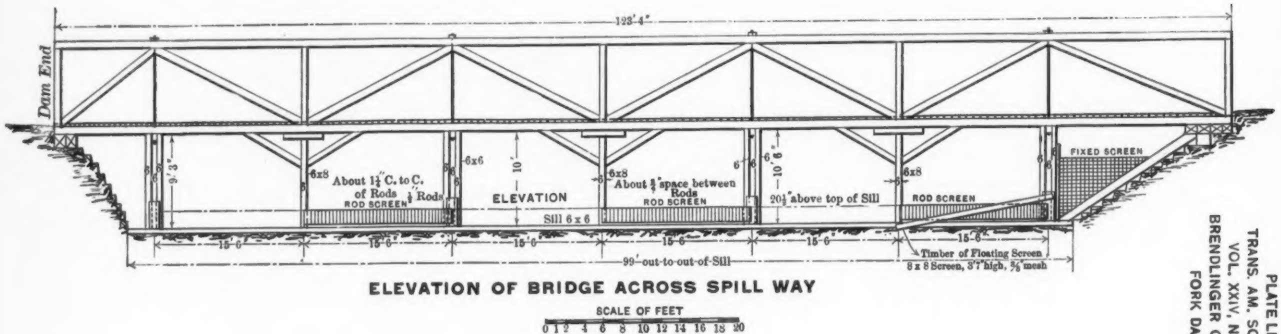
F. Collingwood, M. Am. Soc. C. E.—I did not propose to take part in this discussion. I took some very careful levels on the dam, checking them in both directions. I made the elevations of the dam, practically the same as the committee made them. I am credibly informed that there were frequent repairs of the road on top of the dam, and all theorizing as to the "missing link" is therefore based on an uncertainty.

There is one other point, which I do not pretend to say is or is not the case. I was told by the Superintendent of the South Fork Fishing Club that the floating fish-screen and the end triangular screen were not in place at the height of the freshet, and that a portion of the gratings was not there. They probably were as shown in Mr. Brendlinger's sketches. The three screens in place were 18 inches high, made of ½-inch rods, with 9 rods to the foot. We ought to rely on the committee's statements rather than on anything obtained subsequently, because when the committee was there all the flood-marks were visible, and both Mr. Brendlinger and myself were there long after the occurrence and the flood-marks were gone. I am very clear from careful measurements I have taken that, with all those fish-screens in at the bottom, the section of the wasteway at the bridge was considerably greater than the smallest section below the bridge. If I remember, the report states that the section below was the controlling point. I append a copy of a photograph of the dam which was taken from the level of the crest before this accident occurred, and it is so nearly level that you cannot see any depression. To my mind, after a very careful examination of the whole surface of the crest, I decided that there could not have been any very great depression in the center; not only from what I saw, but what was told me. One intelligent man who was engaged at the dam, at the

PLATE LIII.
TRANS. AM. SOC. C. E.
VOL. XXIV, No. 477.
BRENDLINGER ON SOUTH
FORK DAM.

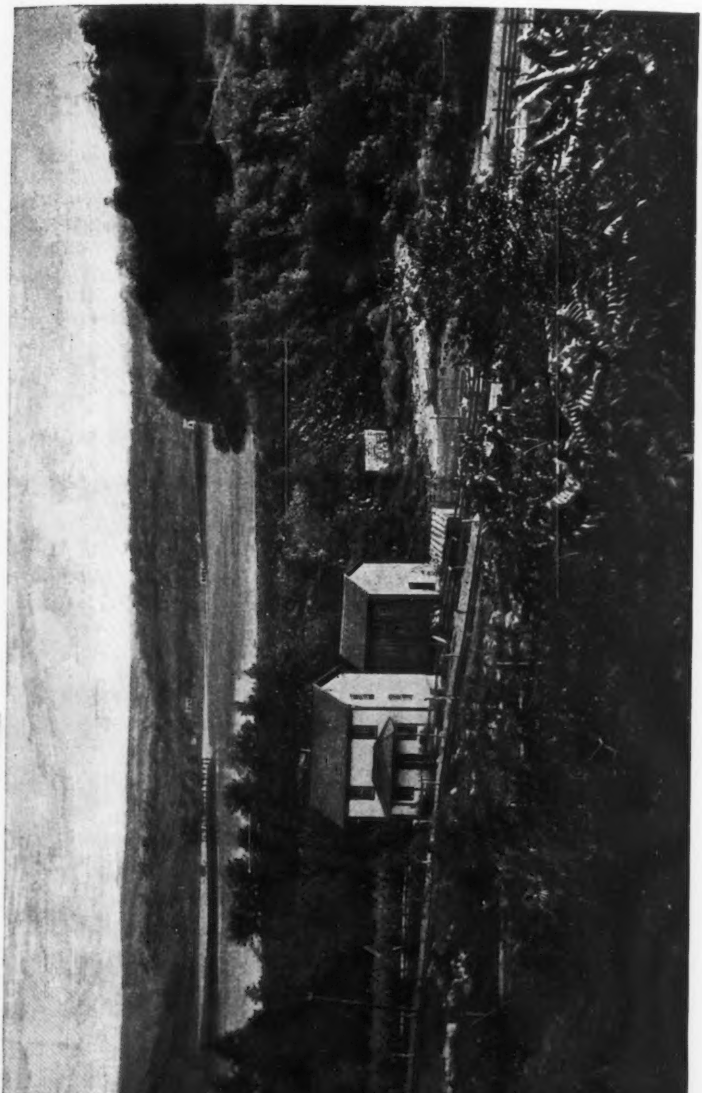


LONGITUDINAL SECTION THROUGH CENTER LINE OF DAM



ELEVATION OF BRIDGE ACROSS SPILL WAY

PLATE LIIIA.
TRANS. AM. SOC. C. E.
VOL. XXIV, No. 477.
COLLINGWOOD ON SOUTH
FORK DAM.



time of the disaster, said that there was a very general crossing over; that they were crossing to and fro through this water for a long time after the water began to go over, wading through the water for a length of some 300 feet. There could not have been any very great depression, as it did not come over their boots. A careful reading of Mr. Park's report fully bears out the committee's conclusions. The statement is clear that the water flowed over at numerous points over a length of 300 feet before it became a sheet, and that it began flowing through the new trench first.

HENRY T. GOLDMARK, M. Am. Soc. C. E.—I have not been at the dam since the accident occurred, and I think perhaps more people have seen the place since than before the disaster. I spent two summers in Johnstown, in 1884 and 1885, and happened to go to this place some half dozen times, and I am very sure that there was no marked depression in the roadway across the dam. We climbed around there a good deal. We were quite a little impressed by the way the waste-weir was obstructed, but that there was any marked depression in the dam never struck us at all.

MR. BRENDLINGER.—If there was a uniform slope, as I have projected it, then a curve would be as nearly correct as could be made. I plotted all these levels out, and I got a curve plotted very nicely. Because an embankment looks level to the eye does not go to say that it is level when you put the level on it. There is a regular drop on both sides of the gap toward the center. It is natural that the dam should be lower at the center than at these points, for the simple reason that this new material was put in in 1880, and, of course, the greatest settlement will be where the deepest part of your embankment is. Of course, you can keep that raised. Why it was not done I do not know.

MR. COLLINGWOOD.—In answer to that I will state another point which was stated to me. I had it from two or three persons that the little furrow that was thrown up by a plow on top of the dam, the water did not begin going over at all until it went over at many places at the same time. That furrow was very hard to plow; the elevation, as we saw it, was some 4 inches only, but it had been worn down by rains. The water did not go over at the center first, but at many places all along its length. If that is so there could not have been any great difference in level. Taking all fair evidence, I could not find any indication of any very great depression.

As to the credibility of reports a year after an occurrence, I was told by a carpenter, who was employed at the cottages, that he saw the water about 20 inches deep on a sidewalk there. Levels taken from this point to the high water mark at the dam showed a fall of over two feet in about a mile. This, in a lake of such dimensions, is incredible.

As proof beyond gainsaying of the magnitude of the flood, all the bath houses and a handsome boat house were utterly wrecked, and a

dam near the mouth of one of the streams forming the lake went out with such a flood that the proprietor could not tell when it happened, it was completely drowned out. It had stood there for sixty years. The proprietor also stated that a pail, which was empty the night betore, had 6½ inches of water collected in it from the rain during the night. This was at a point about two miles above the dam.

JAMES B. FRANCIS, Past President Am. Soc. C. E.—As to the depression, the dam had been lowered about 2 feet to make the roadway, near the ends there were ascents to the level of the top of the dam. On each side of the breach there were slight depressions in the roadway undoubtedly due to corresponding settlements due to the breach. These several points being connected, would give a line substantially a curve, which continued across the breach, would indicate as suggested, a considerable depression at the middle of the breach, but the form of the curve depends on the depressions on the sides of the breach which did not exist until the breach was made, and the curve is consequently no proof of any original depression, although it appears from other sources that there was some.

Mr. COLLINGWOOD.—You estimate there was a depression of 2 feet below the original height?

Mr. FRANCIS.—Yes. The top of the dam was originally 10 feet wide, which was too narrow for a roadway, and the dam was cut down 2 feet.

As to this matter of the fish guard obstructing the flow of water in the wasteway, it did so to a considerable extent, when the depth of water flowing in the wasteway was smaller. In high stages of the water, the controlling point was in the wasteway about 176 feet below the fish guard; at the fish guard it was over 100 feet wide. While at 176 feet from the fish guard it was only 70 feet wide. The bed of the wasteway for the first 176 feet was about level. At the point where it was about 70 feet wide it commenced to fall, and this was taken as the controlling point. When the fish guard was in action, there were floats above the fixed part, which rose and fell with the water. One of these I saw after the flood. During the flood the attempt was made to pull away the fish guard, but with not much success.

Mr. BRENDLINGER.—Did you make a plan or cross-section at the bridge? The elevation of the bridge with the screens?

Mr. FRANCIS.—Yes; I measured that myself and it is taken into account in the calculations.

Mr. BRENDLINGER.—In regard to the screens, I may say that there were three screens in position and one or two below the bridge, lying there. It looked to me as though they were forced out of place by the water; one end of the wire screens is still there.

As to that waste weir, I made it 190 feet to the narrowest point of the spillway from the upper end of the bridge, 55 feet wide at the base and on the right-hand side a slope of 2 feet, then 10 feet from there to

the rock, that made 65 feet wide when the water might have had an elevation of 4 feet. On the other side there was a one to one slope, which of course gave a wider slope.

The top of the dam I made 10 feet wide on the north side; I suppose it is 15 feet wide on the south side.

Mr. Francis.—As the slope below 176 feet from the fish guard was amply sufficient to maintain the velocity, we did not consider that any small inequalities in the section would affect the flow from the lake.

Mr. Brendlinger.—That was below; 190 feet from the upper end of the bridge.

Mr. Francis.—The point I speak of is 176 feet, not 190 feet down stream and that is where it began to pitch off.

Mr. Brendlinger.—I made it 190, you make it 176.

AMERICAN SOCIETY OF CIVIL ENGINEERS.

INSTITUTED 1852.

TRANSACTIONS.

NOTE.—This Society is not responsible, as a body, for the facts and opinions advanced in any of its publications.

478.

(Vol. XXIV.—June, 1891.)

CHARACTERISTICS OF THE RAVINE DU SUD IN THE ISLAND OF HAYTI, AND PLAN FOR AVERTING ITS OVERFLOW.

By J. FOSTER CROWELL, M. Am. Soc. C. E.

WITH DISCUSSION.

The writer recently had occasion to make an extended examination and study of the Ravine du Sud, having been retained by the Haytian Government as Consulting Engineer to devise means of relief for the City of Aux Cayes against further occurrences of the disastrous floods of the past.

The word ravine is here to be taken in its French significance, implying a raging torrent and not merely as a term of topographical configuration. This ravine is a typical tropic "torrentiele," with strongly marked characteristics, and furnishes an interesting object-lesson in the phenomena of sporadic rainfall, for the investigation of which there are few opportunities.

The floods with which we are generally more familiar in our own country are either periodic, such as are augmented by melting snows, or caused by ice gorges, or due to long-continued rains; or else they are accidental, like the Johnstown flood, or the freshets are in streams

that have been modified by artificial dams and other obstructions or encroachments. For the first class, nature has generally provided adequate channels, and the engineer in dealing with them has only to avoid interference. The second class is unlooked for, as a matter of course; consequently, when great floods do occur, they receive only momentary attention, and in degree proportioned to the calamity.

The tropic floods, however, are neither periodic nor accidental. They occur sometimes with great frequency, and again at intervals of years. While always to be looked for at certain seasons of the year, they cannot be foretold, and they come with surprising swiftness and irresistible power. It must not be understood, however, that this describes a condition that can be correctly styled "tropical." In different localities in the same latitude, with apparently similar prevalent conditions, there will sometimes be found wide variations both in rainfall and in temperature, that to the ordinary observer are inexplicable. Of course there are fixed laws, unalterable in operation, but their system is too complex for him who runs to read.

Among the most interesting indices of the vast variants in the matter of rainfall are the so-called torrential streams, which are, in fact, not streams at all, but immense dry courses provided by nature for exigencies that may occur, only at long intervals of years, or repeatedly in some one season; such channels are not peculiar to the tropics, but are found in any mountain region, where there are great fluctuations in the rainfall. The islands of Hayti and San Domingo contain some marked specimens of these "torrentieles," and furnishes a clear exposition of their function.

An examination of the map of Hayti, shows that nearly all the rivers of continual flowage are on the northern sides of the several mountain ranges. The western end of the island is almost entirely mountainous, with a comparatively very small area of plain, so that it may be said to consist of two lofty ranges running east and west, separated by the Gulf of Gonaives. For their height, these mountains are very narrow at the base. Their southern slopes are short and steep and receive little rain, even though the precipitation directly upon and along the southern coast line may at the same time be very great. This may be in part attributed to the fact, that the temperature on the southern slopes is normally higher than that on the northern, which are longer and not so exposed to the sun-action. When the south winds blow, the currents of warm,

moisture-laden air from the Carribean Sea are deflected upward where they strike the heated mountain flanks, and drop little of their load there; but on the cooler northern slope, precipitation occurs very much more frequently, and rivers of considerable size are kept supplied. When the colder winds blow from the north, there is obviously less moisture and consequently less rain. It is quite clear, then, the above being granted, that to produce heavy rainfall on the southern slopes there would have to be:

a. Lowering of the temperature of the mountain surface by unusually cold air currents, aided by the interruption of the sun's rays because of continual cloudy skies.

b. Sudden change of winds to warm water-bearers from the Carribean.

c. Violent electrical disturbances to cause sudden precipitation.

Now, any of these three causes is likely to occur frequently in any season, but their coincidences may be extremely rare. Any one of the three would produce rain in greater or less quantity, but it is only when all act together, probably, that the terrific floods occur which bring the torrentieles into play. This culmination, too, would likely result in setting up for the time being new conditions of temperature, and so the result would not be momentary, like the "cloud-bursts" of the temperate regions, but, when produced, would continue with gradually decreasing energy until the normal was once more restored. And this in fact is what follows when a tropic flood "lets go."

On the southern coast of Hayti is the City of Aux Cayes, the second town and port in importance. The Ravine due Sud is the course of a torrential stream rising in the La Hotte range, and discharging into the Bay of Cayes. A number of branches, none of them inconsiderable in capacity, unite near Camp Perin, about 30 kilometres from the coast, and flow as one river to the sea. Their fall above the junction is very steep, and the waters attain immense velocity. During most of the year the quantity of water is very small, but in the rainy season the flow reaches large proportions. Infrequently, there are enormous freshets occurring irregularly, at intervals of years, and resulting from the culmination of the causes which have already been treated of. The quantity of water passing at such times is incalculable, excepting by carefully measuring the probable maximum, as witnessed by those parts of the ravine which bear clear evidence of having been filled to the brim, but

of never having been overflowed. There is no data of rainfall at hand and no means of securing a record. It must be quite evident, however, that under the conditions cited, no record would be of scientific value that did not cover the flood times and specialize the cloud-bursts in all the tributary territory. Equally obviously, the flood to be guarded against is the extreme condition, and the unknown quantity to be computed is the aggregation of waters in the lower ravine. The method of computation as accomplished in this case will be given hereafter.

Immediately below the confluence at Camp Perin and extending down for several kilometres, the ravine bed is at places nearly 300 metres (or about 1 000 feet in width, varying from that to 150 metres); the banks there are formed chiefly of gravel which has been cut out vertically by the torrent and carried down stream; the coarser material, nearly to Les Cayes, the finer sand and clay, still further to the sea. The bed is covered with large rounded stones from 3 to 6 decimetres (1 to 2 feet) in diameter, that have been rolled down from the mountains by the force of the current. The general level of the banks above the bed is usually about 2½ metres, though occasionally cliffs 10 metres or more in height are met. The stream cuts the width it needs, from time to time, without apparently exceeding the usual flood level.

In the dry season, the flow of water in the lower reaches of the ravine is much less than here at Camp Perin; which somewhat paradoxical condition, is due partly to excessive evaporation, but chiefly to percolation into the gravelly formation; this element is, however, too small in effect to be considered in time of freshets. This compartment of the ravine, from which the sand and gravel have been carried to the sea to make room for the larger rolling stones from the mountains, extends as far down as Carrefour Fond frede (Plate LIV). Next below that, to the crossing of the Route Drouet, there is a compartment that has been scoured out but not refilled, because the velocity of the torrent, diminishing with distance from the mountains and as the more level country is reached, is no longer sufficient to transport the large stones. Down to this point, the work of the torrent is confined to its own bed; there is so far no trace of any overflowing of the banks, and traditions speak of such only vaguely. Nature has thus provided a means of checking the velocity of the torrent and storing up the waters for slower discharge onward to the sea.

For a short distance below the Route Drouet, the unimpaired banks show clearly that the equilibrium has been reached, and indicate that if a uniform channel had extended thence to the mouth, all the water of any flood would pass off in time, harmlessly. But below Drouet the existing channel, unfortunately, only continues to be of sufficient width as far as the Route Torbeck, distant but 1¼ kilometers; here the present bed suddenly contracts, and a short distance beyond, is not sufficiently wide to discharge the most ordinary flood. Two kilometers further down and for a considerable distance below, the sectional area is only one-half of the adequate water way at Drouet. As a natural and inevitable result, the stream at flood has gone over its banks in the vicinity of the town, not only in the narrower parts but also above them, because the accumulating waters were thus held back, and added to the flood quantity. (See Plate LV.)

Although two memorable floods have occurred in recent years, the first of which ruined the stately Quatre Chemin, while the second tore to pieces the newly constructed engineering works which had been planned to control it, the widening of the lower channel has never been attempted, and the conditions are now the same as formerly. The damage has never been repaired, and the entire city lying low upon the beach, is always subject to the liability of another flood as great or greater than the last, which might work untold havoc.

There is another description of damage, less thrilling than destruction by flood, that has been silently but continuously going on, of which the Ravine du Sud is also the agent, and that is the impairment of the harbor. Light freshets bring down sand and mud and deposit them in the harbor; thus a bar has been gradually formed across the mouth of the ravine, which has at length risen above the surface of the ocean, and, augmented by the wave-action, has during the past season completely shut off access by boats to the river mouth, and the lightermen can no longer take their lighters into the smooth water which used to be their refuge in stormy weather. To repair the damage already done to the harbor will be a matter of difficulty and great expense; but a greater damage menaces the wharf and the business front of the city, which will be inevitable with the next great inundation; for unless steps are taken to avert it, the flood will make a new outlet for itself along the line of least resistance, not through the new bar, but parallel to it across the root of the wharf. It is apparent, then, that it is vitally necessary for

the preservation of the harbor, as well as for the security and prosperity of the town and its inhabitants, that the ravine should be given an artificial outlet elsewhere than in the harbor; and, furthermore, the closing up of the present mouth also threatens the health of the city, by cutting off entirely the exit for the drainage that finds its way into the already stagnant waters of the River Renaud. The means of relief which have been recommended are, briefly, to excavate an enlarged channel from the route Drouet to the sea at a point outside of the harbor, and to protect its banks wherever necessary, with adequate revetements of a permanent character. If it were possible to divert the waters of the Ravine du Sud, entirely or in part, and to lead them to the sea through the channels of other rivers, that would obviously be the best disposition of them so far as Aux Cayes is concerned, at least from the flood standpoint, although it would leave the question of public health to be met in some other way. With the possibility of such a diversion in view, careful and extended instrumental examinations of the relations of other streams were made, to ascertain positively whether such a scheme was practicable. The feasibility of erecting a dam, with gates, across the ravine just above Drouet, thus creating a large reservoir for impounding the flood-water and letting it off gradually, had also been taken into consideration; but this idea was abandoned as being both precarious for the present and ineffectual in the future, as well as very costly. To build it would simply be creating a trap for posterity. The plan of diverting the waters seemed more promising, and the hopes of being able to do so were enhanced by the supposed practicability of joining the upper Ravine du Sud with the valleys of the Ravine Seche and the River Torbeck; in fact, tradition pointed to this actually having been done by nature in times past. But the cold logic of facts brought out by the surveys, has demonstrated that this is entirely impossible, for both those valleys are higher than the Ravine du Sud, and are already insufficient for their own discharge. To tap them would be enormously costly, and the only result would be to drain them into La Sud. It is, therefore, an unavoidable conclusion that the present course of these waters must be adhered to, and the works of improvement must be directed toward increasing the width of outlet channel and providing a new mouth.

The problem first to be solved was: "Given the greatest flood that can possibly occur in the ravine, what are the dimensions of the new

channel which is to lead it harmlessly to the sea?" The difficult parts of nearly all problems are the data, and in this case the data had to be constructed. There had been no flood for several years; might not be another for a still longer period; there was no record of rainfall nearer than Port au Prince, and it was not applicable. Even if it could have been accepted as a general guide, to use it would have necessitated measuring superficially various water-sheds contributory to the ravine, covering large areas of steep mountain sides and gulches, practically inaccessible and almost impenetrable. A survey in a balloon might have accomplished it, but available means could not. But nature makes her own records, and in this case had done more than that—she had left her water-meter behind her.

If the reader will glance again at the map, and recur to what has been said about the form of the ravine, he will perceive that above Drouet the bed spreads out like the swell of a bottle's shoulder, while below, it is the narrower neck. Still lower down, the banks are ragged and torn and widened; that is, where the discharge through the neck of the bottle has overfilled a too shallow basin. This neck of the bottle, or this nozzle, if you please, has always heretofore been capacious enough for the moving flood, and if there has been at any time (and, doubtless, such times have been many), more water than could pass through, the excess has quietly stayed in the bottle, which is large enough and to spare. Inferentially this will continue to be so, and the neck of the bottle is the measure of the future flood.

There being, as has been stated, no sufficient data of rainfall or of observed water-marks of the stream at flood, the maximum discharge was arrived at by carefully gauging the capacity of a portion of the ravine at the head of the proposed works, a short distance south of Drouet, about 2¼ miles from the sea. The portion selected was 600 feet in length, it was very uniform in cross-section, with vertical banks, level bottom, no deposits of boulders or silt, and no indication of overflow on the banks. Reliable testimony from several sources corroborated the assumption that the banks had never been inundated, even at the time of the greatest, which was also the most recent, flood. While on the other hand the clean-cut banks showed clearly that their level had been very nearly if not quite reached, and this also was confirmed by the testimony of eye-witnesses and by the flood-marks further down the stream. This compartment, therefore, furnishes in itself the most authentic and

comprehensive measure of the maximum discharge. From averages of a number of cross-sections taken in this compartment, a typical section was constructed, for which the wet perimeter and hydraulic mean depth were deduced. The fall of the surface of the hypothetical flood was carefully determined.

These values were then applied to Stevenson's formula, in which the discharge is a function of the hydraulic mean depth and the fall. It may be convenient to state the formula as follows:

$$X = Y \sqrt{A F}$$
$$Z = \frac{X \times 5\,280}{60} = 88\,X$$
$$D = S\,Z.$$

In which

$X =$ mean velocity of the whole cross-section in miles per hour.

$Y =$ is a frictional co-efficient which is found to vary from 0.65 for small streams under 2 000 cubic feet per minute, to 0.9 for large rivers, but which is here taken as unity to insure the largest probable maximum.

$Z =$ mean velocity in feet per minute.

$A =$ hydraulic mean depth in feet.

$F =$ fall in feet per mile.

$S =$ area of section in square feet.

$D =$ discharge in cubic feet per minute.

Below the point at which the typical section is applied the fall of the natural surface is at the rate of 26.5 feet per mile for 2 000 meters, 17 feet per mile for the next 1 000 meters, and 15 feet per mile thence to the sea. Applying the same formula, but giving Y a value of 0.75, we find the respective areas of channel required to pass D. ($D =$ in this case 500 000 cubic feet per minute.)

In order to minimize the velocity of flood in the new channels, the wet perimeters are increased beyond that of the typical cross-section, and to an extent to make the required depth in new channel, approximately 5 feet throughout. This also makes it feasible to construct the greater part of the new channel entirely below the level of the plain, thus securing an important element of safety; but it is contemplated to obtain a large margin of available section by banking the excavated material above the level of the plain, setting the embankments well

back from the edge of channel and planting them with protecting vegetation. In several localities where the flood surface will necessarily be above the plain, and at others where curved alignment is unavoidable, the banks are to be protected with imperishable revetements of beton, encased with a structure of creosoted timber. Several types of the proposed construction, to suit varying conditions, are shown in the accompanying drawings (Plate LVI). The cost of these revetements is very great per unit of length, but their aggregate length being comparatively small, the expense is justified by the close approach to absolute security at the vital points.

The location sheet (Plate LV) shows two alternative plans; the preferable location is the one more remote from the harbor, but its cost is considerably greater. The improvement comprehends also the question of drainage of the town into the new channels, and the filling up of the bed of the stagnant River Renaud.

DISCUSSION.

Mr. Rudolph Hering, M. Am. Soc. C. E.—In order to find the dimensions of a new channel, which was to lead the greatest flood discharge harmlessly to the sea, Mr. Crowell uses Stephenson's formula, which determines the mean velocity of water in a given section by the mean hydraulic depth, the slope and a certain co-efficient given by its author. Having some misgivings regarding the application of the formula for the case in hand, I made some examinations into the result given by it, as compared with those given by others, and also with actual gaugings made in rivers located on the northern slope of the Alps, having irregular beds of gravel and high velocities, and resembling in their physical character the Ravine du Sud, with cross-sections and slopes both smaller and greater than those of the same ravine. It may be interesting to note this information, and to observe that the Stephenson formula and others give discharges largely in excess of what may be expected from actual gaugings of similar streams.

The following are the various formulas and values of co-efficients suggested by engineers for large and rapid rivers, or for more general application, which, therefore, include the same. In these formulas (feet and seconds) R = hydraulic mean depth. S = sine of slope. c = co-efficient. v = mean velocity.

The fundamental formula used by all authors is that of Chezy namely:

$$v = c\sqrt{RS}.$$


PLAINE À PITRE
RIVIERE ROCOU
TUFFET
RIVIERE BOUCAN
CAMP PERIN
MONTAGNE DE MACAYA
RIVIERE BLANCHE

PLATE LIV
TRANS. AM. SOC. CIV. ENG'RS.
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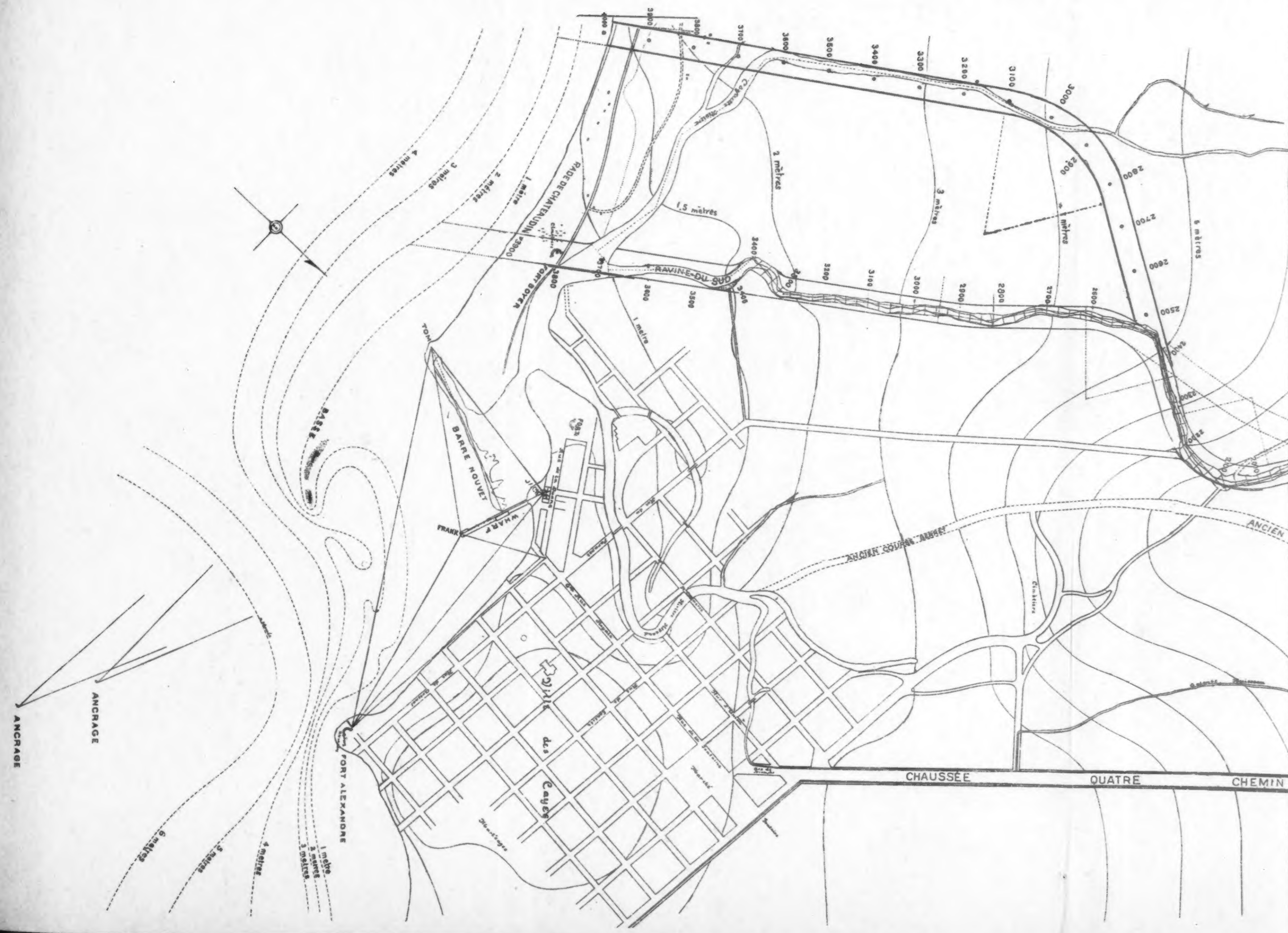
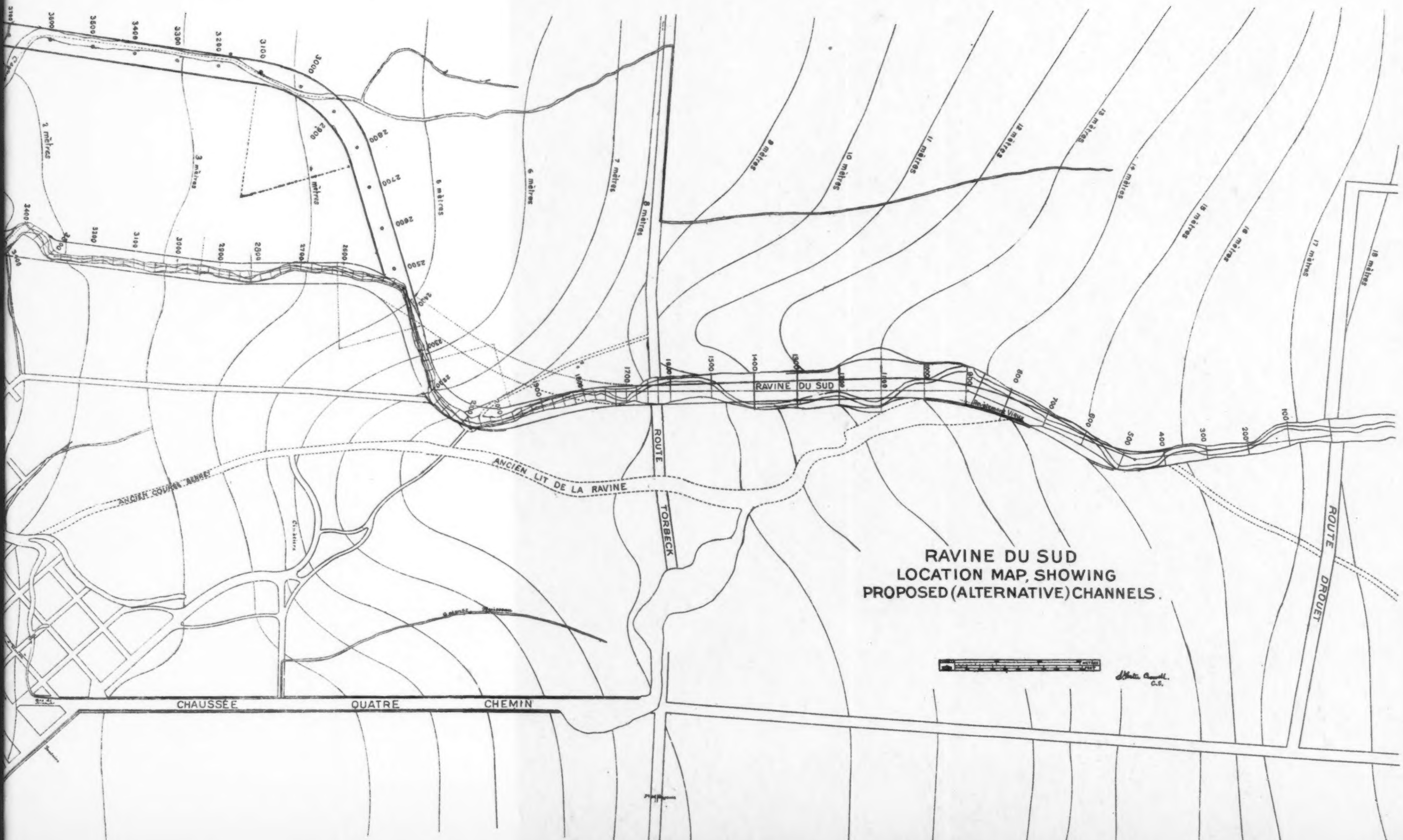
CROWELL ON
RAVINE DU SUD.

GENERAL MAP OF
THE RAVINE DU SUD
ISLAND OF HAYTI

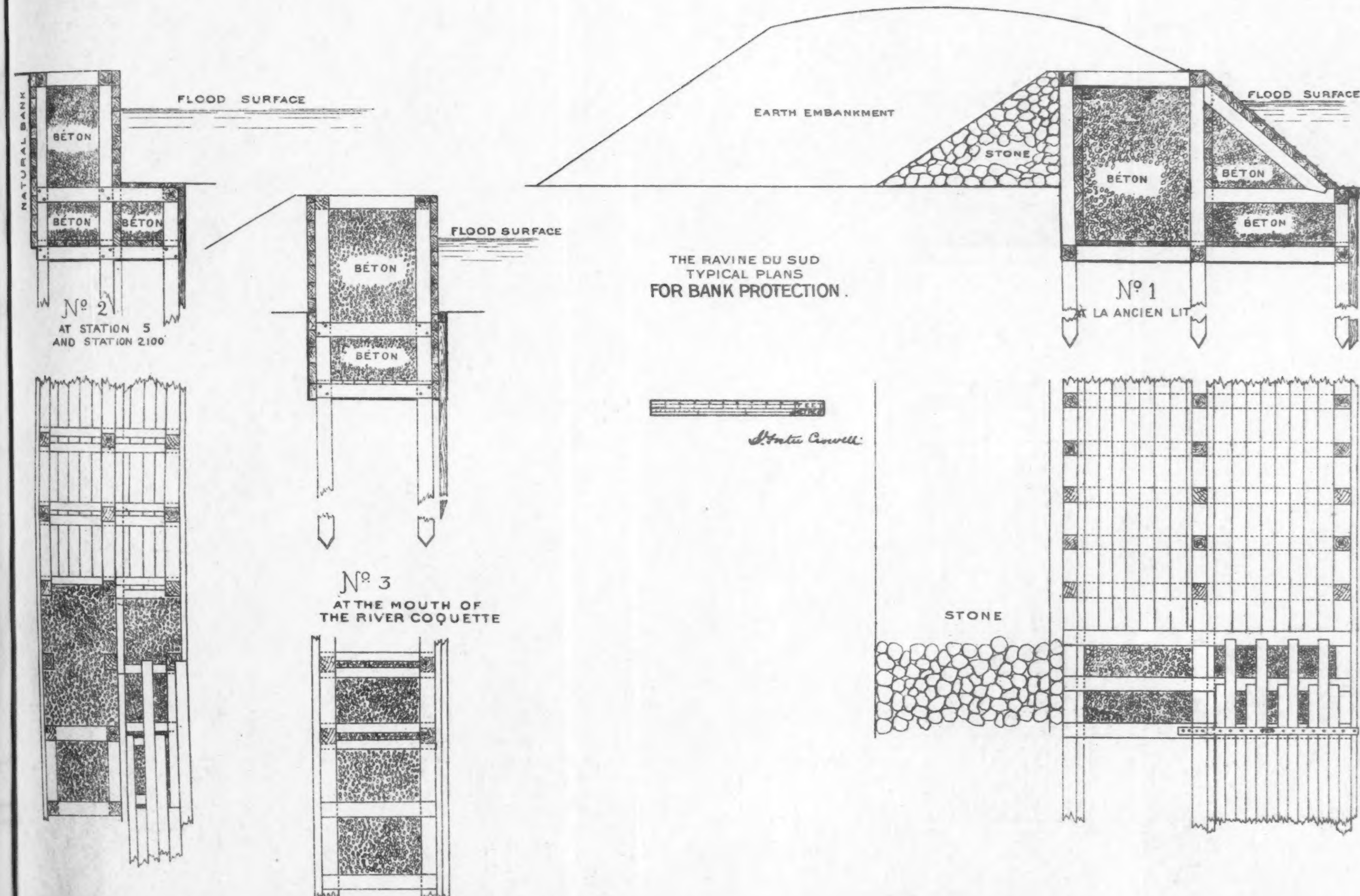


RADE DE CHATEAUDIN
RAVINE-DU-SUD
FORT BOYER
BARRE
BARRE NOUVET
WHARF
FORT ALEXANDRE
ANCRAGE
ANCRAGE
Ville des Cayes
ANCIEN COURANT
ANCIEN
CHAUSSÉE QUATRE CHEMIN
1 mètre
2 mètres
3 mètres
4 mètres
5 mètres
6 mètres
1,5 mètres



RAVINE DU SUD
LOCATION MAP, SHOWING
PROPOSED (ALTERNATIVE) CHANNELS.

PLATE LVI
TRANS. AM. SOC. CIV. ENG'RS.
VOL. XXIV. Nº 478
CROWELL ON
RAVINE DU SUD.



THE RAVINE DU SUD
TYPICAL PLANS
FOR BANK PROTECTION.

J. Foster Crowell

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The values of the co-efficient c, as given by them, are as follows:

First.—Constant values for c:

Young (large rivers)............................$c = 84.3$
Eytelwein (rivers)............................$c = 92.2$
Neville (straight and rapid rivers)................$c = 93.3$
Beardmore (rivers)............................$c = 94.2$
D'Aubuisson (rapid velocities)..................$c = 95.6$
Stephenson (large streams).....................$c = 96.0$
Taylor, Downing and Leslie (large and rapid rivers)....................................$c = 100.0$

Second.—Variable values for c:

Humphreys and Abbot (simplified)..............$c = \dfrac{9.67}{\sqrt[4]{S}}$

Darcy and Bazin (not intended for large rivers) $c = \sqrt{\dfrac{1}{\alpha + \dfrac{\beta}{R}}}$

For earth $\alpha = .000085$, and $\beta = .00035$.

Ganguillet and Kutter.....$c = \dfrac{41.6 + \dfrac{1.811}{n} + \dfrac{0.00281}{S}}{1 + \left(41.6 + \dfrac{0.00281}{S}\right)\dfrac{n}{\sqrt{R}}}$

Table No. 1 gives the numerical results obtained from the above formulas for the co-efficient c, for the mean velocity v, and for the discharge. The author, Mr. Crowell, has kindly furnished me with the following elements to enable these calculations to be made:

Hydraulic Elements of the Ravine du Sud.

R (A in Mr. Crowell's paper) = hydraulic mean depth = 5.5 feet.
Sectional area of typical cross-section = 569 square feet.
Wet perimeter of typical cross-section = 103.5 feet.
S = sine of flood slope = .00322.

Mr. Crowell further describes the bed as being smooth and level, clean and covered with small gravel. The sides are nearly vertical and smooth.

TABLE No. 1.

Formulas of —		Co-efficient c.	Mean Velocity v.	Discharge D
Young,	Constant values of c.	84.3	11.21	6378.49
Eytelwein,		92.2	12.26	6975.94
Neville,		93.3	12.41	7061.29
Beardmore,		94.2	12.53	7129.57
D'Aubuisson,		95.6	12.71	7231.99
Stephenson,		96.0	12.77	7266.13
Taylor,		100.0	13.30	7567.70
Stevenson,*		106.6	14.18	8069.90
Humphreys and Abbott,	Variable values of c.	40.61	5.40	3072.60
Darcey and Bazin,		82.03	10.91	6207.79
Ganguillet and Kutter,		56.29	7.49	4261.81

* As modified by Mr. Crowell.

Mr. Crowell's description of the character of the bed being smooth and level, with clean and small gravel, is hardly consistent with the velocity he obtains by Stevenson's formula, namely, 14.18 feet per second. Such a velocity would carry along small gravel in suspension. This description, therefore, I think does not characterize the condition of the ravine during the time of the freshet, but only when the flow is very slight.

I will now give a table (No. 2) and corresponding diagram, which show results of actual gaugings in similar channels, near the foot of the Alps. Fortunately these gaugings give elements both larger and smaller than the case in question. We therefore have much assurance that our interpolations are not far from the truth.

TABLE No. 2.

GAUGING OF RIVERS ON THE NORTHERN SLOPE OF THE ALPS. (See "Flow of Water in Rivers," etc. By E. GANGUILLET and W. R. KUTTER. JOHN WILEY & SONS, 1889.)

DESIGNATION ON DIAGRAM.	NAME.	MEASURED.			COMPUTED.	
		R	S	v	c	n
a	Salzach	7.39	1.12	5.79	63.4	.0337
b	Aar, near Thalgut	7.06	1.78	6.77	60.5	.0348
c	Rhine at Bale	6.89	1.22	6.38	69.7	.0300
d	Isar, Bavaria	6.05	2.50	7.18	58.4	.0352
e	Salzach	4.26	1.80	5.15	58.8	.0326
f	Limmat, near Zurich	3.16	2.75	5.35	57.4	.0313
g	Saare, Canton Berne	2.70	3.33	4.56	48.1	.0360
h	Rhine, in Canton Graubunden	2.92	6.00	4.23	31.9	.0485
i	Rhine, in the Domleschger Valley	2.95	7.96	7.42	48.3	.0366
j	Kander, Canton Berne	4.12	9.18	8.69	44.7	.0430
k	Plessure, near Chur	4.58	9.65	13.75	65.4	.0294
l	Rhine in the Rhine Forest	1.21	14.20	6.03	46.0	.0309
	Average	....	....	....	54.4	.0357
m	Ravine du Sud, Haiti	5.50	3.22	7.49*	56.3	.0360†

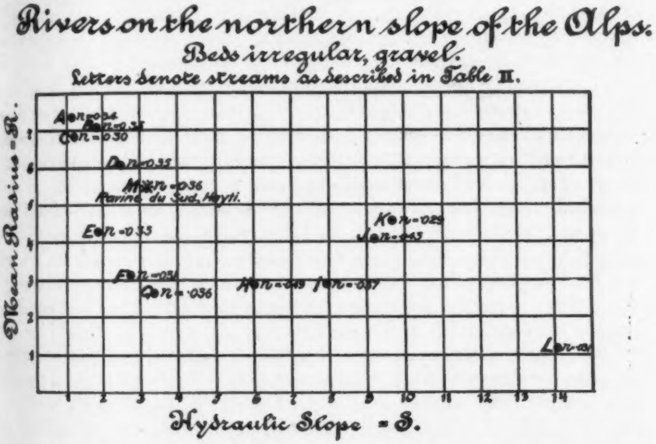
The beds of the above streams are described as consisting of coarse gravel and detritus.

From the measurements of R, S, and v there have been computed the values of the general co-efficient c in Chezy's formula and the co-efficient of roughness n in Kutter's formula. The former vary from 31.9 to 69.7, and average 54.4. The latter vary from .0203 to .0485 and average .0357. Here as in many other cases, it will be observed that c varies much more than n, and that the latter therefore offers a corresponding advantage when an assumption is to be made for a new case. I have assumed it in round numbers at .036 and from it obtained for the Ravine

* Assumed. † Computed.

du Sud, $c = 56.27$ and $v = 7.49$. Stevenson's formula, as modified by Mr. Crowell, respectively gives $c = 106.6$ and $v = 14.18$. The coefficients directly determined from the above gaugings, if applied to this ravine, therefore differ almost 100 per cent. from those of Stephenson's formula. In other words, according to this formula the channel of the Ravine du Sud is supposed to carry almost twice as much water as the qantity calculated from Alpine gaugings for similar channels.

This case again shows the danger of using formulas beyond the limits of the experiments from which they were derived.



W. E. Merrill, M. Am. Soc. C. E., writes:—The problem presented at Les Cayes is similar in character to those that have recently occupied so much attention in the French Alps, and similar effects on a small scale can be observed where streams flowing from the hills of our western river valleys suddenly emerge into the flat bottom land through which the rivers meander. It is a well-known hydraulic law that when a stream, heavily laden with matter in suspension, changes its velocity, a certain amount of this material is dropped, and similarly when a velocity, that is just sufficient to carry down stones, becomes lessened, the stones cease to travel. There is such a marked likeness between the phenomena exhibited by the Ravine du Sud and those reported by French engineers, that it may be worth while to compare the two cases.

According to the latter a mountain torrent always exhibits three different phases: First.—The receiving basin, which is funnel-shaped with various irregularities, and terminates at the bottom in a neck or spout. Second.—The discharge channel or gorge, whose features are less strongly marked than the other two sections, but which is noteworthy

as exhibiting neither scour nor deposit. Third.—The area of deposit, where the washings are arranged according to regular laws; it lies at the mouth of the gorge, and has the shape of a hill or flattened cone of gentle slope.

This description of a torrent in the French Alps is exactly paralleled by that given by Mr. Crowell of the Ravine du Sud, except that, owing to the violence of tropical storms, the upper part of the discharge channel or gorge is filled with rocks and the width is correspondingly increased. The receiving basin is the area above Camp Perin, the rocky part of the discharge channel extends from Camp Perin to Carrefour Fondfrede, the unchanging port of the discharge channel extends from Carrefour Fondfrede to the crossing of the Route Torbeck, and the area of deposit extends from the latter point to the sea.

The specially objectionable section of a mountain torrent is always the cone of deposit, as this occurs where land is susceptible of cultivation, and where overflows are destructive. It is here that the stream ought to be regulated and brought between insubmersible banks. After years of study the French engineers came to the conclusion that it was practically impossible to regulate the lower section of a torrent as long as an unlimited supply of traveling material came from above; moreover the mountains were being denuded, the soil and loose rock were washing down on the plains and the violence of the floods was increasing. They therefore determined to begin at the head, by first regulating the torrents in the upper sections.

The first step was to create a number of artificial falls by the construction of masonry dams. These not only destroyed a considerable percentage of the velocity of the flow, but also served as catch basins to collect debris, thus transforming the bed of the stream in the vicinity of a dam from a gulch with precipitous sides into a flat bottom, which would serve as a footing for the side slopes. The next step was to grade and reforest the side slopes of the basin of reception. Finally innumerable low dams, wattlings and plantations were placed, wherever they could cause deposits or check surface washing. The dam across the torrent Riou-Bordeaux has a height above the bed of the stream of 26 feet, with foundations 15 feet deep, so that the total height of masonry is 41 feet. Its length is 274 feet, the thickness at the crown is 10¼ feet, and at the base, 18¾ feet. The apron in front of the dam is 46 feet wide, and the total cost of the dam was $20 000.

The effect of these works was to cut off from the area of deposit the supply of waste material, and, after this was done, it became possible to give the torrent a regular bed across the cone, and to confine it between definite banks. It is stated that after the works above named were completed, it for the first time became practicable to build a bridge over the torrent Riou-Bordeaux where its area of deposit was crossed by a national highway.

Unfortunately it does not seem financially possible to apply these principles to the regulation and correction of the Ravine du Sud, and moreover the physical difficulties are probably greater than those encountered in the Alps. I have no doubt that Mr. Crowell has done all that the means available would permit, but judging from the experience that I have quoted, the disease has only been checked, not cured. Deposits must continue to be made in the lower section of the ravine, and space must be made for the water either by systematic excavations between the embankments, or by continual increments to their height. The object to be attained, if possible, is to maintain to the sea the velocity of the second section of the ravine; if this were done there would be neither scour nor deposit until the sea is reached.

As the paper does not give a longitudinal profile of the ravine, I have endeavored to make an approximate one from the contour lines shown on the plan of the town and its environs. This would indicate that there was a decided irregularity in the slope of the ravine, which is unfavorable to the conservation of velocity. These conditions seem to show that there is an opportunity to help matters by establishing uniformity of grade, which might be done by excavating the high parts of the bed, or by building levees of extra height where the grade was less than the average, trusting to the stream to complete the work.

The paper calls attention to the fact that, in addition to protecting the town and surroundings from inundation, it is of the first importance to see that its harbor is not destroyed. The shape of the bar that now closes the port would indicate that the resultant of the littoral currents was to cause a drift of bar material to the east. If this inference be correct, any discharge of detritus west of the town is greatly to be deprecated, and it would be justifiable to incur heavy expenditure to discharge the material into the Islet River. The data contained in the paper are not sufficient to enable a person not familiar with the ground to decide on the practicability of this suggestion, and the subject may have received full consideration, but as the references in the paper are limited to possible diversions into streams lying west of the Ravine du Sud, I have thought it worth while to lay some stress on the desirability of making the Ravine discharge on the east side of the town.

Mr. Franz A. Velschow, C. E., writes:—It seems hardly possible to be able to speak with much certainty about an undertaking like the present without having personally gone over the ground itself or without having more detailed data to go by than those contained in this paper, but I happened some years ago to have to deal with a somewhat similar case of floods on the Island of Viti Leon, Fiji, where the conditions of rainfall show the greatest resemblance to those mentioned in this paper. The following discussion of the plans proposed by Mr. Crowell is based on my experience at Fiji.

The project of providing a channel by means of excavations for the

more or less unknown quantities of water which may be expected with floods, seems to me a somewhat unusual and risky measure to take; and the great expenditure it would involve may not be warranted by the advantages obtained. Such a channel could hardly be expected to be of great permanency, but would give rise to constant repairs, owing to the great velocity of the current, unless the whole channel was set in cement or at least very carefully paved. As it is only required to prevent the ravine from overflowing one of its banks, the danger of flood might be averted by protecting the weak points of this bank with properly constructed levees, leaving the flood to take its natural course and spread as much as it pleased on the outside of these levees. That this has been attempted before and without success, may only show that the extent of the floods had been underestimated, and to form anything like an adequate idea of the proper dimensions for these works of protection, it might be highly interesting to inquire about a description of these recently destroyed engineering works. Were they intelligently designed and what were their height and profile?

While the idea of widening the present river bed by excavations may be perfectly justified by the peculiar circumstances of the case, although I can hardly think so, it is when Mr. Crowell maintains the necessity of giving the ravine an outlet into the sea different from what it has now that I find some serious objections to his project. The object of changing the outlet of the ravine or river (a measure which is always a precarious one and apt to lead to most unexpected complications, as only too many engineers are able to testify by their own personal experience), is a two-fold one: (1) to prevent the harbor from silting up by sediment brought down by the ravine so as to form a bar across it such as the "Barre Nouvel;" and (2) to prevent future floods from cutting a passage through the root of the wharf and thereby menacing the business front of the city.

We have here before us the case of a wharf having been built at the mouth of a river, which wharf afterwards called into existence the town Aux Cayes, as indicated by the very name of this town, and there can be little doubt that the break in the beach line (or the deep water in a comparatively sheltered position which originally caused the wharf to be built in this particular place as a convenient place for shipping), has been caused and maintained by the scouring effect of the waters from the ravine. If, therefore, we cut off the ravine from the harbor, what will become of this place as a shipping place? Surely the beach will soon be found to stretch uninterruptedly across the harbor; as ocean beaches always have a tendency to do when there is no flow of water from inland to prevent or counteract it, owing to the sands being constantly shifted by the coastal currents. As a result, after the inhabitants of the town for some time have enjoyed security against floods as provided by Mr. Crowell's project, they may some day wake up to another

certainty, namely, that their wharf upon which the *raison d'etre* of the town depends, is standing high and dry on the sea shore. They would, therefore, either have to move the town, or construct another pier right out into the ocean and exposed to the heavy breakers, which would be very expensive indeed, and not afford such conveniences for shipping as a wharf in a more sheltered position affords. In other words, the cutting off of the ravine from the harbor, means to do away with or give up the naturally favorable conditions for shipping which the place formerly possessed. It is maintained that the Barre Nouvel is being formed of sediments from the ravine, but even if this could be proved actually to be the case, it does not follow that such a bar would not be formed whenever the flow of water from the ravine ceased; and the shape and location of this bar shows that it is not a river bar, properly speaking, but a reef or an ocean bar caused by the shifting effect of the coast currents upon its sands; in fact, this bar appears to be a commencement of the continuation of the sea beach across the outlet, which I have just predicted would take place when the ravine was completely cut off from the harbor.

No doubt, when the wharf was built the deep water caused the current from the land to run in a direction parallel to the wharf. Since then the situation has become slightly altered, and at present it appears from the map that the current is taking a direction in under the wharf, instead of running parallel to it. But it is an easy matter to force the current into its former direction by turning the wharf into a close wall or pier. As a preliminary measure, this could be cheaply effected by driving a row of close-set piles along either side of the wharf. By the current from land being deflected by this obstacle, the scouring would secure deep water along the wharf, and remove the outer or nearest end of the bar, while what remained of this bar would advantageously act as a breakwater. The current from the ravine would be still further guided to advantage by the building of a bulkhead line from the root of the wharf parallel to Rue de la Duane until across the mouth of the Regnaud River, which river or creek of course ought to be filled in, and the drainage of the town led directly into the sea through a system of pipes. The current from the ravine being thus guided, I confess I am not able to imagine how it could endanger the root of the wharf or the business front of the town. On the contrary, what force the current still possessed after having spent itself on the comparatively flat land above the town, I should consider an advantage, instead of a danger, by its scouring effect on the bottom in front of the wharf.

Having hereby briefly advanced my views in opposition to those maintained by the distinguished author of this paper, I should feel happy if they might aid him in finding a possible *juste milieu*, that would satisfactorily solve the difficult problem now before him.

Mr. Crowell also looks into the question of explaining the conditions

of rainfall which cause the floods at Aux Cayes. As already mentioned, the conditions of the Island of Hayti reminds me very much of those of the Fiji Islands, which I had an opportunity of investigating, and this similarity is particularly striking with regard to the conditions of rainfall. By drawing a comparison, we may therefore be able to throw some fresh light on this subject. The Fiji Islands are situated at the same latitude south as Hayti lies north, and both places are mountainous. Now, nearly all mountain islands similarly situated, show the marked peculiarity of possessing what is called a windward side, with superfluity of rainfall nearly all the year round, while the "leeward" side suffers from scarcity or uncertainty of rainfall. Upon examination it has been found that the rainy side of these islands is turned towards the adjacent cyclone track, the location of which again depends upon the location of the great trade-wind centers. As far as Hayti is concerned the cyclone track of the North Atlantic trade-winds center lies generally to the north-northeast of this island, and the consequence is that by far the greater number of cyclones which pass in the neighborhood of Hayti have their course lying north of the island. The prevalence of rain on the northern slopes of Hayti may therefore be explained in conformity to the cyclone theory I have advanced before this Society, and the application of this theory I made with regard to the prevalence of rain on the rising slope of exposed coast mountains.

On very rare occasions, however, the North Atlantic trade-winds' center and with it its cyclone track, moves so far south that the cyclones pass south of Hayti. Then everything is reversed for the time being, and the southern slopes of the mountains receive a heavy rainfall, by cyclone after cyclone striking against them, and that is, in my opinion, the time when the City of Aux Cayes is exposed to the danger of floods, if I am justified in drawing an inference from what I found to be the case in Fiji.

Mr. Crowell's observation that "the southern slopes are short and steep and receive little rain, even though the precipitation directly upon and along the southern coast may at the same time be very great," is full of interest, and may be satisfactorily explained as a logical consequence of my theory. When the current of saturated air strikes against the northern slope it is brought under extra pressure, and therefore yields plenty of rain on this slope. By being forced to move upwards the pressure is decreased, and no rain falls while the rain giving layers of air are at this, for them unnatural high elevation (that is, while above the southern slope of the mountain ridge), the moist air having been forced to a higher elevation than warranted by its inherent specific gravity. After having passed the mountain ridge it sinks. The pressure upon it is thereby increased, and by the reaction which takes place when the air is prevented from sinking any further the pressure is still further increased, and according to my compression theory, we might therefore

expect to find increase of rainfall along the southern coast line or further out at sea, as also observed by Mr. Crowell. See Fig. 1.

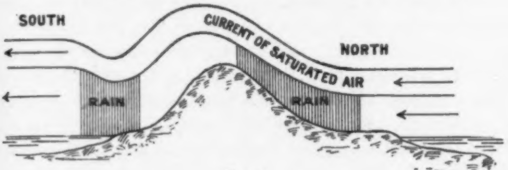


FIG. 1.

On the other hand, the theory set forth by Mr. Crowell that the rainfall on Hayti should be due to moisture collected by the surface air while passing over the 300 or 400 miles wide span of water lying between Hayti and the continent of South America, seem to me impossible to maintain, when it is duly considered that the currents of surface air round the trade-winds' centers pass over thousands and thousands of miles of ocean surface, and in spite of the moisture they may thus collect it never rains within the areas of these centers of high pressure. Let us consider one familiar instance which has direct bearing on this subject. The trade-winds of the North Atlantic pass from the coast of Africa south of the Saragossa Sea and up along the east coast of the United States. When it sometimes happens late in autumn that this current or this center of high pressure advances across the coast line, a period of mild and balmy weather, the so-called Indian summer sets in, and the signal office will predict steady fair weather without rain for New York, as long as these conditions may be expected to prevail. But although it does not rain, the streets become remarkably wet during the commencement of these periods of warm weather. They become covered by a deposit of dew so heavy that even the bright sunshine is hardly able to dry it up until late in the day. The reason is simply this, that the pavement has been cooled down considerably during the preceding period of cold weather, and so dew is deposited when afterwards the current of warm air sweeps over the streets. From considerations like these I should conclude that Mr. Crowell's idea that floods should be caused at Hayti by the moist air from the Caribbean Sea striking against the southern slopes of the mountains which have previously been cooled down by other air-currents, is hardly tenable.

The notion, however, that rain is principally due to moisture collected from the oceans is so ancient and well established, that Mr. Crowell is likely to find a great majority in favor of his theory. It should, in the meantime, be remembered that the evaporation from water-surface is very significant compared to that from cultivated land, and I may finally mention an observation made by Professor Cleveland Abbe in the report of the Chief Signal Officer for 1890, that "the evaporation from salt water is only one-third or one-half that from fresh water." This

observation I have not noticed mentioned any where before, and if it is correct it may be worth while to consider what a difference it would make as to climate in general, if the oceans contained fresh water instead of salt, before we conclude that the evaporation from the surface of the oceans is the main and immediate source of rainfall.

Mr. J. FOSTER CROWELL, M. Am. Soc. C. E., replying to Mr. Hering.—I desire first to thank him for the valuable contribution of the comparative table of formulas for discharge of streams, which comparison cannot fail to be of great service to those of us who in future may be confronted by similar problems; but I do not think his comparison is fair to the Stevenson formula because he presents it not in original form but as modified (by me) for an entirely different purpose than to obtain the amount of discharge, as I shall explain. Without that modification and as stated by Stevenson in his work on rivers,* it would give for the case worked out by Mr. Hering $v = (14.18 \times .65) = 8.2$ for its minimum as against 7.49 by the Kutter formula, a difference of less than 10 per cent. instead of nearly 100 per cent. as when modified. In this connection I would point out anew that under the circumstances cited in the paper it was neither possible nor necessary to know exactly what the total discharge of the stream at flood might be; the desired end being the determination of the dimensions of a new channel, which should have an equal or a greater capacity than the existing channel—the one that was measured—which had proved sufficient for former floods and must continue to be as far as can be known the measure of all future floods. The formula in this case becomes a measure of comparison for channels of different slopes, and I fail to see any danger in the use of the Stevenson formula for this purpose; had the object been to measure the supply of water, neither the method nor the formula would have been permissible. The reason for the modification was to allow for a large frictional effect in the new channels under somewhat slower velocities, but not to minimize the result obtained from the consideration of the present channel; or, in other words, the comparison between the present and the proposed channels is modified in favor of the latter to the extent of 33 per cent. capacity, *ceteræ paribus*.

Certainly extreme caution is called for in dealing, in anticipation, with a flood of unknown magnitude, and the Stevenson formula is consistently conservative from that point of view; and not only so, but the values obtained from it and adopted for the areas of the new channel follow with close conformity the values as deduced from the Kutter formula, as will be quite easy for any one to verify for himself. As a matter of interest I have prepared Table No. 3, giving the two sets of results in parallel columns, using in each the data stated in Mr. Hering's discussion and employing in the Kutter formula the values given by Mr.

* The Principles and Practice of Canal and River Engineering. By David Stevenson, F.R.S.E., Edinburgh, 1872.

Hering in Table No. 2. To make the comparison logical I have added to the Kutter results 25 per cent. after computation. The similarity of the two sets is of course to be expected within the narrow range of velocities in this case, although Mr. Hering's condemnation of the Stevenson formula would predicate very different results. But this is not a battle-ground for formulas, and lest I should be mistaken for an advocate of this class of formulas as against the greater refinement and exactitude of formulas with variable co-efficients, of which the Kutter is undoubtedly the most precise and elegant, I desire to explain that my object is simply to show by the figures that the use of the former is entirely suitable in this case.

TABLE No. 3.—(Crowell.)

Required Channels for the Ravine du Sud.					Cross-sectional Areas.	
Compartment.	Fall in feet per mile.	*R*	*n*	*c*	Values by Kutter Formula.	Values by Crowell's modification (for this particular case) of Stevenson's Formula.
Typical Section..	17.	5.5	.036	56.2	* 569 square feet.	* 569 square feet.
No. 1............	26.5	4.5	.036	53.896	† 660 "	673 "
No. 2............	17.	4.5	.036	53.913	† 824 "	837 "
No. 3............	15.	4.5	.036	53.926	† 872 "	880 "

* Measured. † Increased 25 per cent.

The suggestion, as to the possible diversion of the Ravine du Sud into the Islet River, made by Colonel Merrill, is interesting to the writer as being the plan he was most hopeful of before visiting the locality. There are, however, three separate reasons which I shall proceed to point out, why it is impracticable, each one of which would be potent by itself. In the first place, the littoral current is from the east. Aux Cayes is near the western end of the Bay of Cayes and is protected from the open sea on the south by the Isle de Vache and smaller coral islands, between which and the main land to the westward, is a narrow channel navigable for small ships, but avoided by ocean steamers on account of the direction of the littoral current. Discharging the waters of the Ravine through the Islet would be fatal to the harbor far to the eastward of the limits of the present inquiry. In the second place, the Islet River is a silt-bearer, and while its current is too feeble near its mouth for it to be a serious source of danger to the harbor at present, it is silting up its own narrow bed to an extent to utterly exclude it as an available channel for the Ravine. Thirdly, to direct the Ravine to the Islet would require a large and costly dam across the present Ravine, and extensive dikes and levees at the north end of the Quatre Chemin which has always been the most vulnerable point in the city to floods.

Extensive excavation would be required for a considerable distance and the lower section of the new channel would pass through a salt water marsh.

Replying to Colonel Merrill's other suggestion, a very valuable one, as to the desirability of securing uniform fall for the new channel, I would say that it is unfortunately true that the profile is too hollow to render this possible without diking to a height and extent that could not be justified by either prudence or economy.

Replying to Mr. Velschow, it is not unnatural, considering the absence of detailed information in the original paper as to the harbor conditions, the situation of Aux Cayes with reference to the sea and the chronology of the wharf, that my friend Mr. Velschow should unwittingly have misjudge the indications of the harbor contours, and have been led to the conclusions he advances. As a matter of fact, however, the wharf, a very slight wooden pier on slender piles, and of very modern existence; it having been rendered necessary two or three years ago because the harbor had shoaled so much at and across the mouth of the Ravine du Sud, that the lighters which always previously had been used, could no longer be floated to the beach. The foreign merchants in self-defense built the wharf, and as cheaply as possible. I may remark in passing that it has since been extended to deeper water, for the reason that the bar is making toward the east, and that eventually, unless the present causes are checked, it will have to be abandoned altogether for a new wharf in another part of the harbor, at the east end of the town. In my reply to Colonel Merrill, in this discussion, I have alluded to the situation of the city with reference to the sea, and stated that the littoral drift is from the east—the Bay of Capes is land-locked on the west.

Mr. Velschow further has fallen into the very natural error of supposing the Ravine du Sud in its present position and condition to be a factor in scouring and maintaining the harbor. While this might be true for brief periods during floods and at long intervals, yet usually the force of the feeble current of the Ravine is expended before it reaches the tidal compartment, at a point rather more than half a mile above the *barre nouvelle* and some distance above the confluence with La Coquette, which is a tidal stream. But there are many small freshets having force sufficient to bring the silt into the harbor and deposit it where the stream meets and is overcome by the wave action.

It is, I think, quite apparent that the Ravine cannot be an agent to improve this harbor under any circumstances, and should discharge outside of it. Since the paper was written I have been informed that a small flood actually did cut through the root of the wharf and threatened the business part of the town, as I intimated was possible, but the force of the flood was not sufficient to affect the *barre nouvelle.*

In regard to the primary source of the rain-fall in Hayti, I frankly admit that I may be entirely in error. Whether the heavy clouds which

come rolling in from the Caribbean and accompany the rain, are produced there or are brought from the far northeast by cyclonic action I have no means of verifying or of contesting Mr. Velschow's views; but when we consider the uniformly excessive rain-fall along the west side of the Caribbean, on the Central American coast, and bear in mind the higher temperature of those waters, we cannot easily eliminate them in our minds from rain producers. Moreover, the cyclones blow every year, the southwestern limits of the storm centers of the North Atlantic cyclones are usually in the Bahamas, the south coast rains in Hayti are common, certainly are annual, but the floods are infrequent.

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THE NOZZLE AS AN ACCURATE WATER-METER.

By JOHN R. FREEMAN, M. Am. Soc. C. E.

WITH DISCUSSION.

For measuring the discharge of water in moderate quantity and under considerable pressure a simple smooth nozzle, to which an accurate pressure gauge is properly connected, forms an apparatus of great convenience and remarkable accuracy. For instance, the discharge of a pump delivering its water at a pressure anywhere from 5 pounds per square inch upward, can be metered by this means with an accuracy and convenience not excelled by a weir or by any method known, providing that for the case in question it is allowable to discharge the water in the form of a jet into the air.

The nozzle will, I think, on reflection, be granted to be the most portable and compact gauging apparatus, in proportion to its capacity, which has ever been devised for such purposes. The amount of metal, the complexity of parts, the possibilities for accident, derangement of parts or unnoticed source of error, are in this form all reduced to their lowest terms. The nozzle is, moreover, superior to any sharp-edged orifice, in that its effective diameter can be measured with greater precision, and is less susceptible to wear.

The following paper is devoted mainly to an account of some experiments with certain nozzles up to 2½ inches in diameter, made to determine their suitability for this purpose. The experiments demonstrate:

First.—That the co-efficient of discharge of any particular nozzle may be determined with a very high degree of precision, and is practically constant for all ordinary pressure.

Second.—They indicate that a substantial variation in the method of connecting up the nozzle to its supply pipe would cause little or no change in the co-efficient, and therefore little or no error in measuring the discharge.

Third.—The experiments indicate that to meet any special purpose, one can readily construct a nozzle which is merely a machinist's copy of another nozzle that has been calibrated or tested, and without going to the trouble or expense of calibrating this new nozzle, can safely assume a co-efficient of discharge for it, and with confidence that the possibility for error in this co-efficient is well inside the margin of error or uncertainty incident to most hydraulic measurements.

I advocated the use of the nozzle for this purpose in a paper presented to this Society somewhat more than a year ago, and in a foot note to page 331 of the *Transactions* for 1889, reference was made to a nozzle then under construction for use in gauging the delivery of certain rather small pumping engines. It was in connection with testing this piece of apparatus that most of the experiments described below were made. But it is not to this particular form of the apparatus, but rather to the general subject of nozzle-measurement, that I would direct attention most forcibly.

The sketch, Fig. 1, illustrates the form of device with which these experiments were chiefly made, and which was constructed to serve in certain tests of some small pumping engines. This particular arrangement of nozzle was designed with two objects in view. The first object was to obtain a very portable and accurate water meter of large capacity. The second object was to obtain, if possible, a "Siamese nozzle," so called by firemen, which should deliver a smoother, steadier, and more solid jet, than the Fire Brigade gets from such a piece of its "heavy artillery" when constructed of the ordinary form.

The tests showed this first object accomplished in a most satisfactory manner; but in attaining the second, although fairly successful in producing a jet fully as good and indeed probably a little better than I have seen from any other very large nozzle, it was a less smooth and solid jet than I had hoped for, and there was apparently a somewhat greater tendency of drops to detach themselves from the main stream, than is found

with the best fire nozzles of ordinary size, or of about half the diameter of these large ones.

In devising an instrument for gauging alone, a more simple form, more after the pattern of Fig. 5, would answer equally well. In fact, in my experiments on two pumps I have detached the "Siamese," and merely made use of the nozzle and barrel *C B*, Fig. 1; *B* being simply screwed on to the end of a piece of 4-inch pipe.

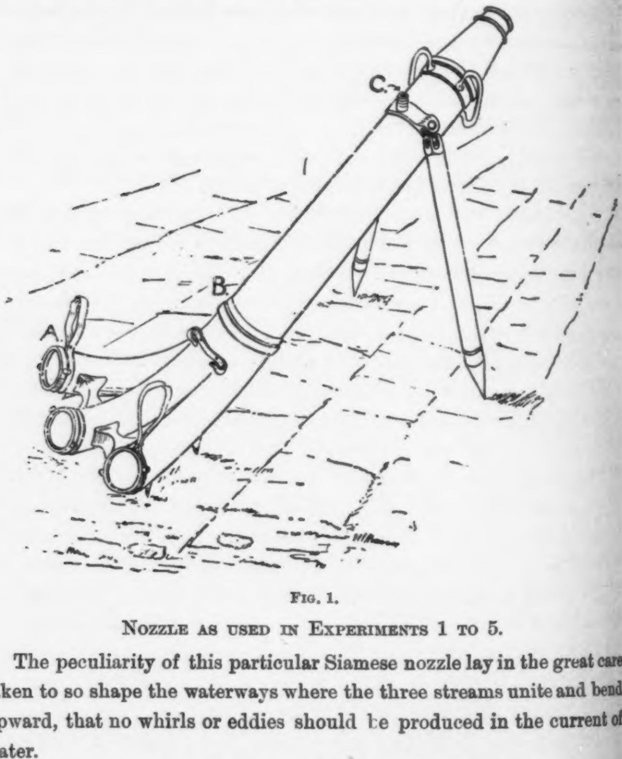

FIG. 1.

NOZZLE AS USED IN EXPERIMENTS 1 TO 5.

The peculiarity of this particular Siamese nozzle lay in the great care taken to so shape the waterways where the three streams unite and bend upward, that no whirls or eddies should be produced in the current of water.

First.—The bends upward are circular arcs carefully made tangent to the axis of the play-pipe, and made plain curves, avoiding even the slightest warping or corkscrew-like twist, lest the water current be thereby set rotating.

Second.—The cylindrical cross-section of the waterway of each of the three branches was carefully merged into a V-shaped section where the branches united near *B*, thus preserving a thin partition of metal between each, so the currents do not unite until the straight single pipe is reached at *B*.

Third.—The area of each waterway was carefully and gradually reduced as the branches came together going toward B, so that the water moves about 15 per cent. faster at B than at A. This gradual convergence tends to prevent eddies in the water at the concave side of the bend.

Fourth.—Hollow cross-connections between the three branches at A were formed in the casting, so that in case one line of hose happened to be delivering more water than the others, the currents would, under the influence of the diminished area at B, tend to equalize, and thus give a more uniformly distributed current in the pipe approaching the nozzle.

Fifth.—To further reduce the liability to twisting of the currents, a thin three-way "rifle-blade," so called, extends from B for 16 inches toward C, this being set so that the blades halve the current from each of the three branches. At C is a hollow ring covering four orifices, and thus forming a piezometer on the same principle as that shown in Fig. 3. The ends at A are fitted with ordinary hose-coupling. Claws on the bottom of the Siamese prevent its kicking back; and two light movable props, which can instantly be folded up out of the way and held alongside the barrel, serve to conveniently support it at any convenient angle, as shown.

Some of the subsequent experiments show this elaborate care in forming the waterways within the Siamese was a needless refinement, so far as the object of accurate gauging of flow was the one in view. They had some little value, however, when the same apparatus was to be used to project a powerful fire-stream.

Description of Apparatus, etc.

These measurements were made in the yard of the Washington Mills Company, at Lawrence, Mass. The Lawrence Water Board, through their superintendent, kindly granted free use of the excellent city supply. This was delivered to the site of the experiments by a 16-inch pipe, and under a pressure of 70 to 75 pounds per square inch. The water was led to the nozzle through from three to four lines of ordinary 2½-inch fire-hose, each about 50 feet in length, which in turn were fed from a large four-way Chapman hydrant. The method of measurement of the rate of discharge was by turning on the water to any desired pressure, regulating this pressure by throttling at the hydrant gate, and meanwhile letting the jet discharge freely into the air. Then after all had got into a condition of steady flow, by a slight swinging * of the nozzle-pipe the jet was turned into the hood, by which it was deflected downward into the tank, and a few moments later a reversal of this swinging motion threw the jet free from the tank and into the air beyond.

The jet delivered was caught, and its gallons per minute measured in a rigid and tight rectangular tank, 5 x 8 x 4½ feet deep, constructed

* In cases where the iron cylinder shown in Fig. 5 was used, this was so heavy that it was necessary to keep it stationary, and therefore a light, quick-moving, swinging hood was devised.

of plank and lined with sheet zinc with soldered joints.* The tank set up above the ground level, blocked up on heavy timbers, and together with its drain-pipe, was frequently inspected throughout. It was thus not possible for leakage or waste to escape unnoticed. The horizontal sectional area at various depths was carefully measured, and although the dilation of the tank, due to the pressure of its contents when full, was so small as to be almost inappreciable in the final results, it was measured and compensated for in the effort to guard against error.

The depth was noted by two observers independently, before and after each "fill," by means of a glass tube large enough to avoid error from capillarity, which rested against a very accurately divided scale attached to the side of the tank; and the error in measuring depth of fill could hardly exceed two one-hundredths of an inch, and was probably seldom more than half this.

In manipulating these powerful jets there was sometimes a little water lost by splashing back as the stream was turned into the hood over the tank (this was estimated roughly and compensated for), but as the whole amount so splashed out seldom amounted to more than one-tenth of 1 per cent. of the volume under measurement, there could be no noteworthy error from this source. The area of the tank at various depths was measured independently by different observers.

It would appear that the limit of error in measuring the number of gallons could not be greater than one gallon per thousand or one-tenth of 1 per cent. The length of time occupied by filling the tank to any depth is the element where errors of observation would have the greatest weight; or since in general the duration of fill was only about a minute and two-thirds, or 100 seconds, an error of a single second in measuring this time would have effected the result 1 per cent. At first we relied wholly on a stop-watch for this measurement, and as the observers were skillful, it was thought the error of a single determination need not exceed a quarter of a second, and that the average of a series might be considerably closer. I had some misgivings, however, and so borrowed two other good stop-watches, and in several experiments the same fill was timed by three observers. The nozzle was always swung with a very rapid jerk, and thus the beginning and end of

* This tank was the same illustrated in Figs. 3 and 4, page 315 of the *Transactions* Am. Soc. C. E. for 1889; and the mercury pressure gauge mentioned a little later on is the same as illustrated in Fig. 1, page 308, *idem*.

the experiment were as sharply defined to the eye and ear as could be desired, by the "swish" of the water striking the edge of the hood. But there were occasional unaccountable discrepancies of half a second, or even more, which led me to perfect an arrangement with an automatic chronograph, by which the time was measured with mechanical accuracy to about the fiftieth part of a second.

The operation of this device was beautiful in its perfection and certainty, and a glance at the wonderful manner in which, for instance, Experiments 55 to 62, page 9, repeat the same value for co-efficient, gives ample proof of its satisfactory work.

The electrical contacts were not perfect when we first started the chronograph, or for experiments up to No. 44, although occasionally it served. These contacts and break-circuits were overhauled on September 27th, and after that time (Experiment 46) were perfectly satisfactory.

For the main part of this chronographic apparatus, I was indebted to my friend Mr. William F. Sherman, agent of the Atlantic Cotton Mills, for whom it was devised by Mr. T. G. Estes and originally set up for use in some steam-engine experiments. It consisted principally of a marine chronometer beating seconds, to which Mr. Estes had applied an electric break-circuit, so that in connection with two or three battery cells it would beat seconds on the index of an old-style Morse telegraph recording instrument, and thus as the paper ribbon ran over this Morse recorder, it was indented with a series of dashes at distances of nearly an inch apart along the ribbon and at intervals of exactly a second apart in time. Beside this chronometer index on the Morse recorder another arm or index for marking the ribbon was placed. This additional arm was operated by an electro-magnet, actuated by another pair of battery cells, whenever the circuit was closed by the contacting of two copper plates, one attached to the nozzle, and both so set as to come in contact when the jet from the nozzle was just halved by the edge of the hood leading the water into the tank.

The record made by the chronograph was recorded on the ribbon thus:

Seconds............ ——— ——— ——— ———

Nozzle movement... — —

and by dividers and scales this space was readily converted into time, probably to within one-fiftieth of a second.

MEASUREMENT OF PRESSURES.

These were in every case measured by an open mercury column; and the correction to apply for elevation of center of each orifice above datum of gauge, was determined by filling hose and nozzle full up to center of the orifice, but with no current flowing meanwhile, and then noting reading of gauge. Slight oscillations of the water pressure were continually taking place while the nozzle was discharging, but the variations of the gauges were seldom greater than one-quarter pound between successive observations; and the extreme variation during a whole experiment was very rarely a pound. The ordinary fluctuation was perhaps about half as great as the limits just stated.

The pressure gauges were read each half minute.

Taking all known sources of error into consideration, I do not think it reasonable to suppose that the error in determination of pressure on piezometer could exceed one-half of 1 per cent.; and from a study of the apparatus see no reason why, in the ordinary run of experiments, it could have been over a fifth of 1 per cent.; but on computing the final results, some slight irregularities were developed, which perhaps may have come in through the pressure measurement. The resulting error in co-efficient, which varies as the square root of the pressure, would be only half this per cent. of error in pressure.

The piezometer, or orifice, through which the pressure near the base of the nozzle was communicated to the pressure gauge, when the apparatus was used in form shown in Figs. 1 or 4, consisted of a series of holes of $\frac{1}{8}$ inch diameter, all lying equidistant from each other in a circle around the inside of the pipe. These holes were drilled and finished with care; were normal to the axis, and their inner edges were free from any burr or projection into the pipe. All communicated into a channel $\frac{1}{2}$ inch square encircling the pipe, and which in turn communicated with a $\frac{1}{4}$-inch pipe from which a piece of rubber tubing led to the pressure gauge. Care was taken to free this connecting tube from air bubbles.

The diameters of the three nozzles were calipered by three observers. Thus:

First.—The machinist tried to construct them of the exact nominal diameter.

Second.—They were calipered by means of a Vernier caliper, divided into decimals of a foot.

Third.—The nozzles were calipered independently by another assistant, using a new Brown & Sharpe micrometer Vernier caliper, divided into thousandths of an inch. The resulting diameters were:

Nominal diameter (inches).......	1.750	2.000	2.500
First caliper....................	1.748	1.998	2.500
Second caliper..................	1.748	1.998	2.499

The error in value used, therefore, did not exceed a thousandth of an inch.

The temperature of the water during these experiments was about 69 degrees Fahr. on August 25th, and 64 degrees Fahr. on September 27th.

The various experiments are given in the following table, and are grouped so as to bring all with apparatus of similar form together. The serial numbers will serve to identify their actual sequence. It should be stated that the 2-inch nozzle was the size governing the design of the whole apparatus shown in Fig. 1. The 2½-inch nozzle was devised and added subsequently, as much with a view to pushing the velocity of approach and the velocity past piezometer to an extreme limit, and seeing if they would still serve, as for any other purpose. In other words, the 2½-inch and the 1¾-inch nozzles were not supposed to be so well suited for this special piece of apparatus as the 2-inch, but were devised partly for seeing if the convergence of the cone could be increased or diminished to this very considerable extent, without affecting the value of the co-efficient.

The result was even more favorable than expected.

As still another modification, in order to push the approach of the current into an extreme condition and observe the effect on the co-efficient, we tried a few experiments with the apparatus in the form shown by Fig. 5, except that the tin cone was removed and the water entered the nozzle past the square corner of the brass flange on the head of the pipe. The experiments were made and worked up the same in all respects as those in the tables following.

The results of these three experiments:

Observed pressure, pounds, square inch,	52.04	51.88	20.24
Co-efficient discharge, 2-inch nozzle	0.9870	0.9850	0.9897

Of course it was to be expected that the contraction past the sharp

TABLE No. 1.

Experiments on Discharge of 2-inch Nozzle.

	Serial No. of Expt.	Duration of Fill. (Seconds.)	Observer of time.	Gallons per minute measured in tank.	Pressure shown by gauge at base of nozzle. (Lbs. sq. in.)	Proportional correction to compensate for velocity past piezometer.	Total (static) head on orifice. (Feet.)	Theoretic velocity due corrected head.	Co-efficient of discharge.	Average.	
Aug. 25th, 3 to 5 P.M.	1	85.7	*F*	798.4	41.90	1.0839	104.87	82.13	0.995	.995	Nozzle connected to play-pipe and Siamese, in ordinary normal condition, as shown in Fig. 1. Pressure gauge connected to piezometer at *C*. In Experiments 1 to 5 there were 3 lines of hose leading into Siamese. In Experiment No. 6, 4 lines of hose were used.
	2	104.9	*F*	671.1	29.37	"	73.50	68.76	0.999		
	3	100.7	*F*	665.7	29.34	"	73.43	68.73	0.991		
	4	145.4	*F*	477.4	14.88	"	37.24	48.94	0.998		
	5	139.2	*F*	474.2	14.88	"	37.24	48.94	0.992		
	6	84.7	*F*	848.0	47.48	"	118.83	87.43	0.993		
	23	89.04 88.9 88.5	Chrn. *B* *P*	760.6	38.36	"	96.00	78.58	0.990		Same as Experiment 6, except that wire screen was inserted at *B*, to remove eddies, or stray irregular currents, if any existed.
Aug. 28th, 4 to 5 P.M.	24	89.0 89.0	*B* *P*	770.6	38.53	"	96.42	78.75	1.001	1.0000	Arrangement same as in Experiments 1 to 6, except that the longitudinal diaphragms or "rifle blades" were removed. The screw at *B* was left in as in Experiment 23.
	25	87.53 87.4	Chrn. *B*	769.7	38.52	"	96.40	78.75	1.000		
	26	95.8 95.8	*B* *P*	608.8	24.16	"	60.46	62.36	.999		
Aug. 29th, 4 to 5 P.M.	41	72.0 71.6 71.9	*F* *P* *B*	861.0	48.14	"	120.85	88.17	.999	.998	Nozzle connected to pipe and Siamese, in ordinary normal condition, except that 4 lines of hose were used, as in Experiment 6 above. The pressures were, however, measured by a different mercury gauge from that used in experiments above.
	42	82.58	Chrn.	777.7	39.49	"	98.83	79.73	.998		
	43	98.6 98.6	*B* *P*	672.6	29.54	"	73.93	68.96	.998		
	44	118.7 118.5 118.8	*F* *B* *P*	540.8	19.06	"	47.20	55.39	.999		

	38	74.6	Chrn.	864.8	48.62	"	121.68	88.47	1.000	.997	Pipe *B C* removed, and nozzle attached directly to Siamese, with only a short piezometer intervening, as shown in Fig. 4.
	39	128.3 128.2	*B* *P*	481.4	15.22	"	38.09	49.50	0.995		
Aug. 29th, 3 P.M.	40	128.6 128.0 127.8	*F* *B* *P*	482.0	15.23	"	38.11	49.51	0.996		
	55	70.45 70.6	Chrn. *F*	850.0	51.45	1.0003	118.84	87.43	0.9950	.9954	The nozzle in these experiments was detached entirely from the pipe *B C*, and also from the Siamese, and was attached to the head of a large cast-iron cylinder, as shown in Fig. 5, with a view to eliminating possibility of error due the high velocity past piezometer in the experiments given above.
	56	72.83 73.0	Chrn. *F*	849.7	51.45	"	118.84	87.43	.9946		
	57	86.16 86.6	Chrn. *F*	749.5	39.85	"	92.04	76.94	.9969		
	58	98.00 97.8	Chrn. *F*	649.1	29.94	"	69.15	66.70	.9960		
Sept. 27th, 2 to 4 P.M.	59	122.14 122.3	Chrn. *F*	533.8	20.26	"	46.80	54.87	.9957		
	60	138.50 138.7	Chrn. *F*	459.4	15.01	"	34.66	47.22	.9957		
	61	165.30 165.0	Chrn. *F*	377.0	10.14	"	23.42	38.81	.9942		
	62	228.45 228.3	Chrn. *F*	265.5	5.03	"	11.60	27.32	.9948		
	63	315.1	*F*	179.9	2.46	"	5.68	19.11	.96±.02		The pressure in this experiment was too small to be measured with great precision by the means at hand.

TABLE No. 2.

Experiments on Discharge of 2½-inch Nozzle.

	Serial No. of Expt.	Duration of Fill. (Seconds.)	Observer of time.	Gallons per minute measured in tank.	Pressure shown by gauge, at base of nozzle. (Lbs. sq. in.)	Proportional correction to compensate for velocity past piezometer.	Total (static) head on orifice. (Feet.)	Theoretic velocity due corrected head.	Co-efficient of discharge.		
Aug. 28th, 2–4 P.M	18	68.6 68.8	*B* *P*	998.9	23.03	1.2341	65.63	64.97	1.005	1.001	Nozzle connected to play-pipe and Siamese in ordinary normal condition, as shown in Fig. 1, except that four lines of hose were used.
	19	67.8 68.1	*B* *P*	995.8	23.01	"	65.57	64.95	1.002		
	20	85.31 85.2 85.5	Chrn. *B* *P*	783.1	14.29	"	40.73	51.19	1.000		
	21	68.4 68.3	*B* *P*	999.7	23.18	"	66.06	65.19	1.002		
	22	70.4 70.6	*B* *P*	963.9	21.64	"	61.67	62.98	1.000		
Aug. 28th, 9–10 A.M.	7	68.4	*F*	996.4	23.49	"	66.94	65.62	0.992	.992	Nozzle connected in same manner as in Experiments 18 to 22, except that a block was subsequently found lodged in one of the branches to Siamese. This obstruction may have caused eddies in currents approaching nozzle.
	8	66.0 66.5	*F* *B*	994.3	23.54	"	67.09	65.69	.989		
	9	89.8 89.5	*F* *B*	763.6	13.80	"	39.33	50.30	.992		
	10	84.4 84.6	*F* *B*	767.5	13.87	"	39.53	50.42	.995		

Aug. 29th, 2 P.M.	33	55.58 55.5 55.5	Chrn. *P* *F*	1154.3	31.25	"	89.05	75.68	.997	.995	Nozzle attached directly to Siamese with only a short piezometer-piece intervening. Pipe *B C*, Fig. 1, being removed, thus leaving apparatus in the form shown in Fig. 4.
	34	55.3 55.2	*F* *P*	1156.0	31.21	"	88.93	75.63	.999		
	35	77.04 77.15 77.0	Chrn. *F* *P*	800.0	15.02	"	42.80	52.47	.996		
	36	62.44 63.2 62.6 62.6	Chrn. *F* *B* *P*	1044.6	25.95	"	73.93	68.96	.990		In this experiment a screen composed of two thicknesses of wire netting was inserted at joint between Siamese and piezometer.
Sept. 27th, 11 A.M.	50	46.15 46.1	Chrn. *B*	1132.0	37.86	1.00075	87.49	75.02	.9863	.987	Nozzle attached to end of a large cast-iron cylinder, as shown in Fig. 5.
	51	49.04 48.9	Chrn. *B*	1130.2	37.64	"	86.98	74.81	.9874		
	52	52.08 52.3	Chrn. *B*	1131.6	37.79	"	87.31	74.94	.9869		
	53	76.34 76.8	Chrn. *B*	820.7	19.91	"	46.00	54.40	.9861		
	54	77.94 77.9	Chrn. *B*	822.2	19.94	"	46.07	54.44	.9872		

TABLE No. 3.

Experiments on Discharge of 1¾-inch Nozzle.

	Serial No. of Expt.	Duration of Fill. (Seconds.)	Observer of time.	Gallons per minute, measured in tank.	Pressure shown by gauge at base of nozzle. (Lbs. sq. in.)	Proportional correction to compensate for velocity past piezometer.	Total (static) head on orifice. (Feet.)	Theoretic velocity due corrected head.	Co-efficient of discharge.		
Aug. 28th, 1 to 2 P.M.	15	{111.2 {111.0	B P	590.2	40.29	1.0474	97.44	79.17	0.996	0.999	Nozzle connected to play-pipe and Siamese in ordinary normal condition, as shown in Fig. 1, with pressure gauge connected to piezometer at *C*, 3 lines of hose leading into Siamese.
	16	{113.1 {113.2	B P	594.1	40.12	"	97.03	79.00	1.005		
	17	{144.0 {143.7	B P	460.4	24.52	"	59.29	61.76	0.996		
Aug. 28th, 11 to 12 A.M.	11	{109.7 {109.7	F B	582.1	38.86	"	93.98	77.75	1.001	1.001	Nozzle connected tbe same as in the three preceding experiments, except that a large block was subsequently found half obstructing one of the three legs of the Siamese, and thus, perhaps, disturbing currents.
	12	{113.3 {113.3	F B	582.5	38.91	"	94.10	77.80	1.001		
	13	{142.5 {142.8	F B	467.2	24.81	"	60.00	62.12	1.005		
	14	200.3	F	350.4	14.16	"	34.25	46.94	0.998		
Aug. 29th, 8 to 9 A.M.	27	{99.5 {99.4	B P	680.2	53.38	"	129.09	91.12	0.998	0.999	Nozzle connected directly to Siamese—the pipe from *C* to *B* being now removed—a different piezometer ring (Fig. 3) from that used in Experiments 11 to 17 is also now in use, being interposed between nozzle and Siamese. 4 lines of hose are now in use instead of 3, a wire screen is interposed at joint between Siamese and piezometer section with a view to reducing the eddies due bend and 4 lines of hose.
	28	{97.6 {97.4	B P	680.5	53.14	"	128.51	90.92	1.000		
	29	{142.9 {143.1	B P	463.5	24.74	"	59.83	62.04	0.999		
	30	{97.2 {97.2	B P	699.6	56.42	"	136.44	93.68	0.998		All circumstances the same as in the 3 observa-

	30	97.2 97.2 97.2	*B* *P* *F*	699.6	56.42	"	136.44	93.68	0.998	0.997	All circumstances the same as in the 3 observations next above, except that wire screen is now removed.
Aug. 29th, 11 to 12 M.	31	149.3 148.9 149.2	*F* *B* *P*	467.2	25.08	"	60.64	62.45	1.000		The approach to nozzle is now so probably disturbed by the change of direction and the fourth line of hose that some variation in co-efficient might be looked for.
	32	152.7 152.1 151.6 151.9	Chrn. *F* *B* *P*	360.6	15.17	"	36.68	48.57	0.9922		
	45	87.6	*B*	689.3	57.48	1.0002	132.76	92.41	0.997	0.995	The nozzle is now disconnected entirely from Siamese and pipe *C B*, and is attached to large iron cylinder, as shown in Fig. 5, with a view to removing any chance of error due to the very high velocity past piezometer in the foregoing cases.
Sept. 27th, 9 to 10 A.M.	46	78.64 78.6	Chrn. *B*	688.0	57.39	......	132.53	92.33	0.9959		
	47	100.6	*B*	576.7	40.23	......	92.91	77.31	0.997		
	48	103.81 103.6	Chrn. *B*	576.1	40.28	......	93.03	77.36	0.9953		
	49	150.0 149.55	*B* Chrn.	406.1	20.14	......	46.51	54.70	0.9924		

corner would diminish the co-efficient; but, as showing the constancy of the co-efficient of discharge of a nozzle under different conditions, it is interesting to see that this difference of full 90 degrees in inclination of surface leading to nozzle, as compared with that when attached to barrel *C B*, affected the co-efficient only about 1 per cent.

The compensation for effect of velocity of approach in reducing piezometer readings, was made according to the formula

$$h = \frac{h_1}{1-C^2\left(\frac{d}{D}\right)^4}.*$$

We can perhaps most expeditiously, as well as most completely, discuss the foregoing tables of the experiments by propounding a few questions, and answering them out of the record of the experiments.

First.—What values for co-efficients of discharge of these three nozzles were deduced from these experiments?

> *Answer.* With apparatus set up, as shown in Fig. 1 or Fig. 5, I finally adopted the value 0.995 for use with each of the three nozzles.

Second.—Can this kind of water meter be relied upon to duplicate its own results?

Is any ordinary accidental variation in method of setting up liable to introduce a change of rate, or error in its indication, while using the co-efficient as previously determined once for all?

> *Ans.* So far as the nozzle itself is concerned, the foregoing experimental values show a remarkable agreement, considering that an effort was made to vary the conditions, and that hardly more than two of the experiments on any page were made under precisely the same conditions as to the method of connection, pressure, etc.
>
> A strong effort was made to vary the conditions, to use now one pressure gauge and then another; to first try one set of piezometric orifices and then the other; to continually vary the pressure on the orifice; to first have the channel approaching the nozzle give the greatest possible parallelism of the fluid veins and then to connect it close to a Siamese, one of whose branches was receiving a double water-supply, and thus undoubtedly causing the water to approach the orifice in a disturbed condition.

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* For the derivation of this formula, see page 456, *Transactions* Am. Soc. C. E., 1889. Also page 476, *idem*.

In the subsequent use of this particular piece of apparatus, as for instance in measuring the delivery of a pump, we may (if our pressure gauge is all right and the apparatus set up in almost any reasonable manner) have the greatest confidence that the error of any series of measurements would not exceed one-half of 1 per cent.

Third.—If a person desiring a circular nozzle for use as a meter should find it expedient to vary the size of the orifice, making it, for instance, three or four or more inches in diameter, and should moreover vary the angle of convergence of the opposite sides anywhere between 10¼ and 15 degrees, would he be justified in using the co-efficient .995?

Ans. If for a few inches back from the end there be given to the interior surface a smoothness equal to that which a good brass finisher gives to ordinary lathe-finished work; and if, moreover, the piezometer orifices are carefully made normal and flush with a surface parallel to the axis of the current, and withal so near the nozzle proper that no loss of pressure of importance can intervene; then if pressure is accurately measured, I see no reason why this co-efficient may not be applied with a feeling of entire confidence that the result is within 1 per cent. of the truth.

Fourth.—Will a single constant value for the co-efficient of discharge of a nozzle apply for all pressures, high as well as low?

Ans. The constancy of the co-efficient of discharge under a great range of pressures is well shown by Experiments 55 to 62.

This series of experiments was probably the best of any in the favorable conditions for extremely accurate work. The water pressure was steady; the swinging hood for deflecting water into tank was working smoothly and without splash, and the chronograph was working at its best.

Starting with about 120 feet head of water, and in the successive experiments gradually reducing this head to only a tenth part as great as that with which we started, the variation in the co-efficient between the first condition and the last was less than one-tenth of 1 per cent. For heads below 10 feet, some variation would very likely occur, but no reason appears why the co-efficient for 200 or even 400 feet head should not be the same as for 100 feet.

It may be asked how the various values for co-efficient deduced by the foregoing experiments agree with those of other experimenters, or if they are not unreasonably near to unity; or in other words, if the necessary friction losses would not prevent the actual discharge from being so extremely near to the theoretical discharge as the experiments indicate.

There are no previous accurate experiments on record on nozzles quite so large as these, under such high pressures, so far as the writer is aware. Hamilton Smith, Jr., describes some experiments* on certain very smoothly finished nozzles of cast-iron, about 1 inch and 1¼ inches in diameter, converging from a base of 4 inches diameter in a length of 12 inches. (Angle 15¼ and 13 degrees.) He found an average value of 1.005† for certain nozzles which diverged very slightly at the extreme end, and a co-efficient of 1.006 for one in which the slightly diverging part was cut off, leaving the nozzle convergent at an angle of about 8½ degrees to the very end.

These were tried under a head of about 145 pounds or 335 feet.

Experiments by the writer on a variety of fire-hoze nozzles 1⅛ inch in diameter, gave a co-efficient for play-pipe and nozzle combined of .977.

Certain experiments by Mr. E. B. Weston on a 1⅛ inch fire-hose nozzle gave for the nozzle and play-pipe combined a co-efficient of .975. Obviously the friction loss in the "play-pipe" might account for a considerable part of this difference between .975 and the value .995 found above. I will confess, however, that when upon computing the results of the experiments of August 25th–27th, or Nos. 1 to 44, I could hardly give credence to a co-efficient so near to unity as those obtained, for obviously there must be considerable absorption of head by friction against the walls of the nozzle. There seemed to be no source open for error, unless indeed the piezometer orifices did not, under the extremely high velocity past them, truly reveal the pressure; or unless perchance, the method followed for correcting for velocity of approach, although apparently sound in theory, contained some error.

I therefore connected the nozzles to a conduit so large in diameter that the velocity past piezometer was so small as to remove all chance for such errors, but still got high values for the co-efficient.

In the experiments on smaller nozzles communicated to this Society last year, I showed by certain experiments on the distribution of velocity within the jet, that the lessening of the co-efficient below unity was mainly due to retardation in those filaments of the jet lying close to the walls of the orifice.

I therefore now attempted the measurement of the velocity at various

* Hamilton Smith, Jr., Hydraulics, p. 207; also p. 285–7.

† Greatest possible limit of error thought to be 3 or 4 per cent.

points between center and circumference of the jet issuing from the 2-inch nozzle, and by the use of the mercury gauges obtained a somewhat greater degree of precision than had been secured in the similar experiments of the year previous. For measuring the velocity the same modification of "Pitot's tube" previously described, was used in a similar manner. Fig. 6 shows the velocities revealed at the various points.

It is shown that the issuing water appears to possess its full theoretic velocity at all points from the center of jet out to within $\frac{1}{7}$th of an inch from the side. From thence outward, retardation, due to the friction of walls of nozzle, occurs; but it will be noted that the total amount of this retardation is small in comparison with the whole volume of curve. Integrating this by means of a large scale plotting, we find that the total effect of this retardation at the side amounts to $\frac{65}{10000}$ of the whole delivery, or, in other words, having thus by "Pitot's tube" measured the retardation near the walls of orifice, then assuming this to be the sole cause in diminishing the discharge below that theoretically due the head, we thus deduce 0.9935 for the co-efficient of discharge of the 2-inch nozzle under 50 pounds pressure. This is a remarkably close agreement with the value determined by the entirely different method of measurement in the tank.

It may next be questioned whether the pressure or head can, in general, be measured with reliable certainty by the piezometers, when, as in the apparatus shown in Fig. 1, with the $2\frac{1}{2}$-inch nozzle attached, the velocity past the piezometers is very swift. (This was nearly 25 feet per second in some experiments.)

Darcy first questioned whether or not the height of a fluid column connected through the medium of an orifice like that of our piezometer, gave a true measure of the pressure within the pipe.

Mr. H. F. Mills set out to answer this question by his elaborate series of experiments on orifices in the side of an open trough, and showed conclusively its accuracy for application to an open trough.

Next, Mr. Hamilton Smith, Jr., in his valuable treatise,* though admitting its value for the trough, doubts the value of its application to pipes for very exact measurements; and my eminent friend, Mr. Herschel, in conversation, once called my attention to the possible uncertainty of compensating the indication of a piezometer, past which the water was moving swiftly, by a correction based on *mean* velocity,

* Hydraulics, page 258.

when the velocity of those layers of the current of water next the piezometric orifice was much less than the said mean velocity.

We therefore are justified in extracting from these experiments such information as we can, bearing on the accuracy of thus applying piezometers to closed pipes under high velocities and heavy pressure. The evidence, though not extensive and perhaps not absolutely conclusive, tends to show their great accuracy of indication, and tends to fully justify their use. This evidence is as follows:

First, take the experiments on the 2-inch nozzle with this attached to the iron cylinder, under conditions where the velocity past the piezometric orifice was, at most, only $1\frac{1}{2}$ feet per second, and the head due this velocity only $\frac{1}{3000}$th part of the total head acting on orifice, and compare it with Experiment No. 1 or No. 6, where the velocity past piezometer is 25 feet per second and the head $\frac{1}{12}$ part of the total head. We may, from the fact that the co-efficients as deduced were subtantially alike, reason with justice that the piezometer, with the very high velocity past it, nevertheless told the truth; and we are also justified in believing that the rule followed in compensating for the influence of this velocity on the reading of the gauge was correct. Moreover, although in Experiments 38 to 40, an entirely different set of piezometer orifices and connections were used, the results were still the same.

With those experiments where the $2\frac{1}{2}$-inch nozzle was used, the conclusions as to the general accuracy of piezometric indication and the correctness of the method of correction are still more striking, for here the velocity past piezometer was as high as 29 feet per second and the correction about $\frac{1}{5}$th of the total head. And here again two different piezometers gave substantially the same result.

Nevertheless, although, under these swift currents past them, the piezometers do give substantially correct results, these same experiments perhaps intimate that the method of correction, or possibly the height of the piezometric column, may have been out of the way (too low) by a half of 1 per cent., or thereabouts.

I also had some apprehensions lest the fact that the sectional area of the conduit opposite the piezometers was slightly greater than a few inches up stream where obstructed by the "rifle blades," might throw the piezometer region a little into the condition of a diverging pipe, and thus tend to slightly lower the piezometric column and give an increased value to the co-efficient. Experiments 24 to 26, made with

these blades removed, as also such experiments as made with the aid of the piezometer close to Siamese, show that any difficulty from this cause was not great.

The question next comes, is this apparatus of a form convenient for use?

As to convenience for transportation, the apparatus shown in Fig. 1 to-day lies packed in a common small trunk 12 x 18 x 30 inches; one man can handle it easily, and can in ten minutes take it out and set it up ready for use. It requires no foundation and can set anywhere.

I have with it gauged a pump delivering 1 500 gallons per minute, or at the rate of 2 000 000 gallons per twenty-four hours, under 75 to 100 pounds pressure. I have also used it in similar experiments on two other pumps of nearly equal size and running rapidly, and have in no case found the pulsations of pump to seriously interfere with its use. Ordinarily, I have fed the water to it from the pump through the medium of three or four lengths of common fire-hose. At other times, I have disconnected the play-pipe from the Siamese, and screwed the play-pipe and nozzle directly on to the end of a line of 4-inch iron pipe. For a gauge I have used a thoroughly first-class Bourdon gauge, and have rated this by comparison with a mercury column, or, more conveniently, a Crosby gauge tester.

To answer a question as to what special form I would recommend as a standard for a nozzle meter, it may be said first, that the details of the apparatus are susceptible to great variation without impairment of the accuracy.

The essentials are:

First.—A smoothly tapering nozzle whose sides converge at an angle of somewhere between 5 and 7½ degrees to the axis, and whose interior is smoothly polished for a distance back, equal to say three or four times the diameter of orifice.

Second.—A waterway leading to the nozzle, so formed that the water shall reach the nozzle undisturbed by violent eddies or swirls. This can easily be secured by a judicious arrangement of screens or gratings near the nozzle, even though the converging pipe be crooked, obstructed or unfavorable.

Third.—A well-made piezometer orifice at base of nozzle carefully made flush in a surface parallel to direction of current.

Fourth.—An accurate pressure gauge connected to piezometer by

pipes absolutely free of leakage; and if the water pressure is subject to oscillation, care should be had that the cocks and pipes leading from piezometer give equal ease of flow in both directions, so that action analogous to that of a ratchet and pawl cannot occur and hold pressure, so as to give the gauge a slightly fictitious reading.

A word of caution may be added, to the effect that even the best Bourdon pressure gauges are not instruments of precision unless frequently compared with a mercury column, or preferably, perhaps, a Crosby gauge tester.

Merely as a study I have also sketched out a design for an apparatus intended to cover a large range of capacities, shown in Fig. 7, and a reference to which will serve to illustrate the points which I think should be kept in mind.

The smaller nozzles can be detached, leaving the larger ones in place, and thus securing an orifice of any desired size. Intermediate sizes can be attached to the same base, as for instance a 2½-inch could be screwed on to the end of the 3-inch, in place of the 2-inch shown. The inside corner at the extreme end of each nozzle in the series should be rounded out to lessen the chance of bruising. A straight cylindrical tip of uniform diameter for a length equal to about half the diameter of nozzle is formed at the end of each, and the angle where this unites with the conical portion is smoothly rounded off. The rubber ring for packing the joints in the nozzles is placed at the back end of the joint and held from blowing out by a lip which projects over it. The three screens are for removing swirls or eddies in the approaching current, and the four sheet metal vanes would serve to prevent any rotary or twisting motion. The whole could be secured to supports sufficient to resist the recoil by attachment under bolt heads "A."

A device like this could be made very light and portable by making the shell of steel plate and the bell mouth of a thin bronze casting. A regulating valve somewhere in the line of pipe to "C" or "B" would serve by partial closing, to throw any desired back pressure upon the pump, and where oscillations of pressure were extreme, an air chamber could be extemporized out of a piece of vertical pipe with advantage, perhaps. Fig. 7 is a somewhat elaborate device to cover a great range of deliveries. Of course, much smaller and simpler devices can be made to serve for special cases.

With the 6-inch nozzle and 75 pounds pressure, this would discharge about 9 000 gallons per minute, or nearly 13 000 000 gallons per twenty-four hours, or 20 cubic feet per second; while with the 2-inch nozzle and 5 pounds pressure, the discharge would be only about 260 gallons per minute.

The nozzle may discharge horizontally, or upward or downward, or at any angle between.

In certain pump tests, when discharging about 1 500 gallons per minute and under about 100 pounds pressure, or at the rate of about 2 000 000 gallons per twenty-four hours, I have with great convenience let the nozzle discharge vertically downward into the pump well.

PLATE LVII
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FREEMAN ON THE
NOZZLE AS A WATER METER.

- Fig. 2 -

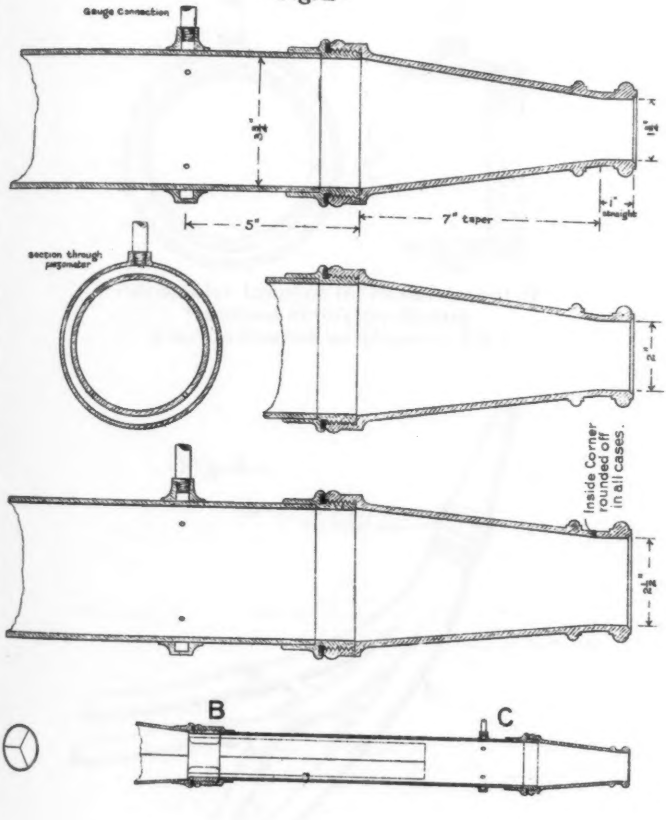

PLATE LVIII
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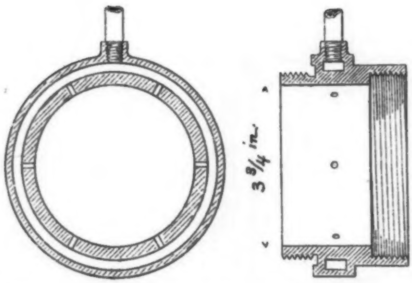
FREEMAN ON THE
NOZZLE AS A WATER METER.

-Fig. 3-



Piezometer Coupling for Measurement of
Pressure acting on Nozzle
when connected as shown in Fig. 4.

-Fig. 4-

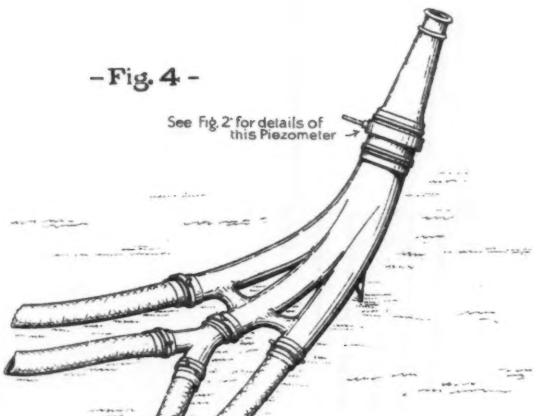


PLATE LIX
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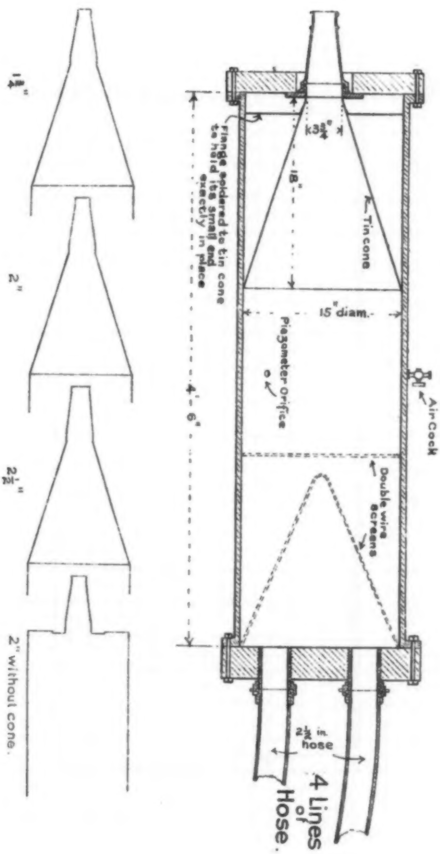

-Fig. 5-

PLATE LX
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— Fig. 6 —

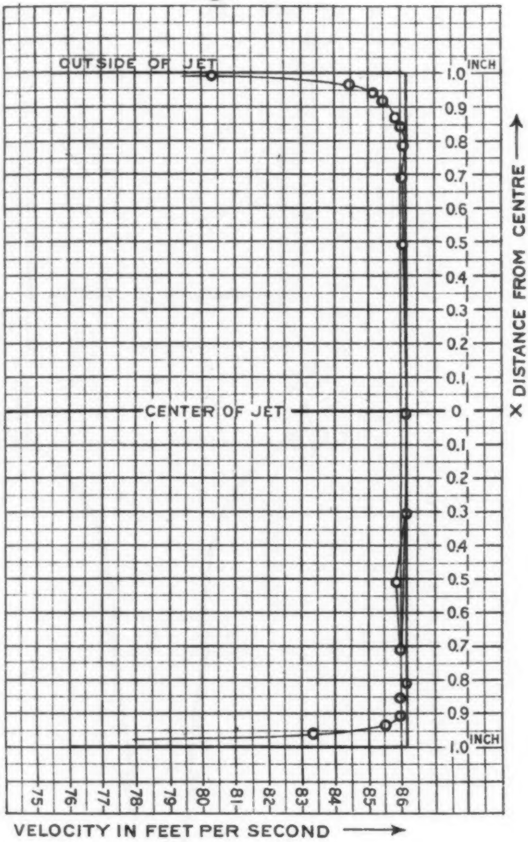

PLATE LXI
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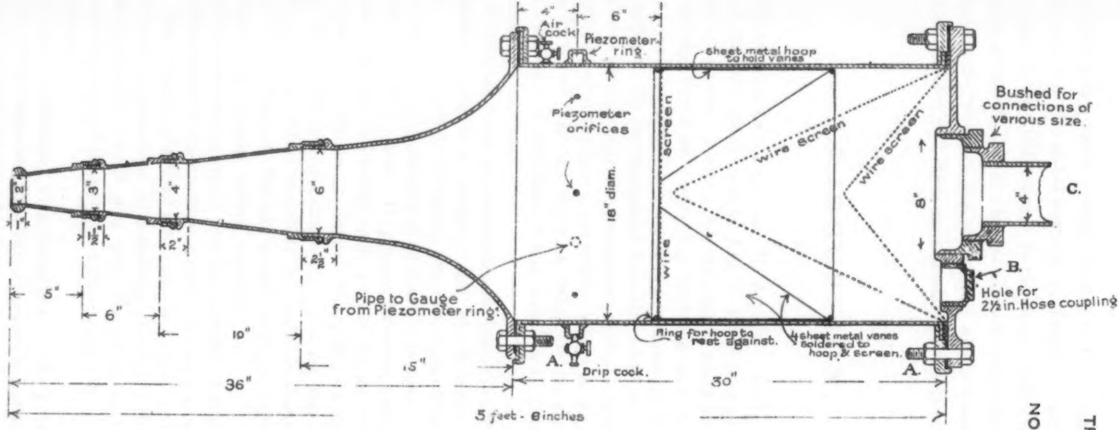
FREEMAN ON THE
NOZZLE AS A WATER METER.



Form suggested for a Meter Nozzle of large capacity.
Fig. 7.

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By placing the nozzle nearly vertical and lowering its end to within an inch or two of the water surface, we found it easy to avoid carrying down much air into the water. If the pump well was not of ample size, and if the contact of the jet with air was not thus reduced to a minimum, the whole of the water in the pump well would quickly have become so full of air bubbles and foam as to greatly interfere with pumping.

DISCUSSION.

WILLIAM BARCLAY PARSONS, M. Am. Soc. C. E.—In the suggested form of variable sized nozzle, Fig. 7, Mr. Freeman appears to violate his first "essential" for an accurate nozzle, when he lays down the rule that a nozzle should be "smoothly tapering and its interior smoothly polished for a distance back equal to say three or four times the diameter of orifice." In the suggested form there will be at each intermediate nozzle two angles, where the horizontal orifice breaks the regularity of the taper, and also the line of separation where each nozzle is screwed on. It will be noted that the tapering sides are regular for a distance of only 2 diameters in the case of the 4-inch and 2-inch nozzles, and 1½ diameters for the 3-inch.

As to whether these variations are sufficient to affect the accuracy of the measurements is a question that Mr. Freeman can undoubtedly answer. The reason for violating one of his principles of design appears to be to make an apparatus that contains a number of nozzles and thus answers all ordinary cases. It appears, however, that the same end might have been attained by having each nozzle separate and complete, and made to bolt on to the main barrel where the 6-inch nozzle is now fastened. While this arrangement would call for more and larger pieces, nevertheless, they could probably be made as cheaply as the proposed system, as the expensive fittings would be done away with, and certainly all possibility of causing eddies avoided.

Mr. Freeman recommends that the corners of the ends of the several nozzles be rounded to lessen the chance of bruising and so making a rough edge. But to be of practical value this device must be used by persons who may not be particular in watching for the little chances of error that are not readily visible, and who, therefore, may not be sure that each nozzle is screwed down to an absolute fit, with no intervening particles or crack.

The CHAIR (A. FTELEY, Vice-President).—The point seems to be well taken; it is to be regretted that Mr. Freeman is not here to answer these questions; I do not doubt but that, as he will see this discussion, we

may have the benefit of his answers at some future time. You may allow the Chair to say a word in regard to the advantage that I believe the profession can derive from this, if Mr. Freeman is correct, in testing pumping engines. We all know that in testing pumping engines of ordinary capacity it is always quite a question to determine exactly the amount of water that is pumped. I have seen cases where the results were only incomplete, because there was no time or no desire on the part of those in charge to make the necessary expenditures to secure perfect gaugings by ordinary means, and if the proposed methods are as accurate as represented by Mr. Freeman, and from his well-known accuracy I hardly have grounds to doubt it, it would certainly be a great advance in the question of perfect tests of pumping apparatus. It would be of great advantage to secure a perfect and cheap apparatus of this kind.

Mr. Freeman (by letter).—In reply to the suggestion or query of Mr. Parsons, as to whether the successive breaks in the continuity of the convergence of the nozzle sketched in Fig. 7 might injuriously affect the accuracy of measurement, I would say that I do not think that this would be the case. The apparatus shown in Fig. 5 contained similar angles, though fewer, but showed no defect or variation of co-efficient when tested. I may add that I sketched the device here in this particular form partly to bring out the propriety of this very thing, and to indicate the extent to which I considered it allowable to go, should special circumstances render this desirable. I should myself, however, always have a little preference for the single, uniform, uninterrupted taper, whenever conveniently attainable. So long as the angle of convergence is kept within the limits there shown, and while there is nowhere a divergence, I see no reason to apprehend that such changes in convergence or that such small crevices at the joints would induce disturbance in the flow by causing eddies.

Charles B. Brush, M. Am. Soc. C. E.—We are all extremely interested in this subject and in the way it has been treated by Mr. Freeman. It is a serious question, however, with me, whether there is any better way of determining the amount of water delivered by a new first-class pump than by plunger displacement. In a test recently made at Memphis, a very carefully constructed weir was prepared, and the amount of pump displacement was checked very carefully, with the records taken every thirty seconds during two hours, one hour in one day, the other hour the next day. The weir was all right with one exception. The extreme desire of the engineer to have the crest perfectly true led him to delay setting the crest, in Portland cement, until the day before the test. The result was that a frost unexpectedly set in that night and left the surface a little ragged and rough around the brass frame of the weir. He was a very conscientious man, and recognizing the fact that his weir was not entirely correct and true, be proposed not to use it in the test. It was

used, however, and the result was that the pump displacement and the weir measurement differed about 2 per cent. During the second night the frost acted still further on this cement, and made it a little more ragged, so as to make a perceptible curve, and we found on the second test a difference of about 3 per cent.

A well constructed pump is really a large meter, and my experience (when I am sure of the pump as to workmanship, size, stroke, etc.) leads me to receive with a great deal of confidence the records obtained from displacement; and I frankly confess, with more confidence than I do the results obtained from the apparatus proposed by Mr. Freeman. Of course, after a time, the wear of the pump will affect its accuracy, but I am speaking of a new, well-built pump, in good order. In neither the case of the nozzle proposed by Mr. Freeman, nor the Venturi meter proposed by Mr. Herschel, both of which to my mind are based on the same general principle, has there been any practical results obtained outside of their own experiments. I have personally urged that the Venturi meter should be set up and actually tested on a line of water-pipe in regular service. So far as I know, none have yet been tested in that way. I am quite anxious to see it done, and to see the results after considerable periods, and a comparison of the results thus obtained. In relation to Mr. Freeman's apparatus, also, I believe there are none in practical use. I am so much interested in it that I propose to put one up, and in doing so I shall call upon Mr. Freeman to assist me in having it constructed as he thinks it ought to be. It will be attached on a plant near New York, and when in operation I shall be happy to have the members come over and see it. I want to see this thing actually working, not as an experiment, but running for a considerable period of time and under different conditions, such as we usually have on water-works. If we can really rely upon it within 2 or 3 per cent., we have something that is very simple, that is very accurate, and that is very valuable in connection with water-plant operations.

It is extremely important, however, both in the case of the Venturi meter and this nozzle meter that we should be able to get at results quickly and with ordinary men, such as we employ around pumping engines, rather than to employ a corps of scientific men to determine results. If an ordinary engineer who is running an engine at a pumping station cannot operate these meters then they are practically useless. The recording attachment should be so simple that an ordinary engineer can take the record from it, and an ordinary calculator can obtain the results. If that can be obtained it is certainly very valuable and very important.

The Chair (A. Fteley).—Nothing can be more gratifying than to see a member offer to practically advance the question by proposing an actual test, but his statement that the action of pumps is so reliable as to make

of them exact meters is not supported by facts. In estimating the delivery of a pumping engine it becomes necessary to determine as exactly as possible the accuracy of its performance. If the loss of action is 2 or 3 per cent., and if the nozzle meter, which would be very inexpensive, will give an absolute test within 1 per cent., it would certainly be a gain; and as to the necessity for having skilled labor in order to ascertain what the work is, it seems to me that it would be as easy for ordinary engineers to attend to these tests as it would be upon an ordinary test of a pumping engine.

Mr. Brush.—It is my opinion that the pumps to which I have referred recorded correctly the actual discharge. The idea I wished to convey was that the weir was within 2 or 3 per cent. of being correct, notwithstanding its faulty condition.

The Chair.—I understood that.

Mr. Brush.—All weir measurements which depend on a series of questions as to what are proper constants to be used, I rely on less than I do on the accurate measurement of the pump displacement. In relation to the error of 1 per cent., as found by Mr. Freeman, I think the extreme care that was required to obtain this close result might not be observed in ordinary work; and therefore we might not get to so small an error generally. I do not consider the value of these meters to consist alone, or even in greater part, in the fineness of the test of an engine during its first use. I consider their value lies in their power to determine the increase of the slip of the pump. I believe a pump can be constructed so that its record may be relied upon in the beginning, but I do not believe in this record two or three years later. I know there is a slip, and I know that slip increases. I want to know from time to time the amount of this increase. If this apparatus of Mr. Freeman is all right I cannot see any reason why it should not be as correct two or three years from the time it is first attached as in the beginning. If we can obtain correct results on pipes delivering from 2 000 000 gallons up to 13 000 000 gallons per day, the results should be true up to 200 000 000 gallons per day, and therefore we may be able to ascertain accurately how much water is delivered in large conduits.

Mr. Freeman (by letter).—In reply to the suggestions of Mr. Brush that a well constructed pump is of itself, while new, a high grade water-meter, I most heartily concur that this is generally true. There may, although but seldom, be grave exceptions, however, and as Mr. Brush has stated nearly at the close of his remarks, the slip of a pump is likely to increase with age. My friend Mr. Coggeshall, Superintendent of the New Bedford Water Works, stated a good instance of this a year or two ago. As I now recall it, they had been making some computations of the cost per million gallons pumped with one of their engines. They figured the gallons by plunger displacement, and the result showed a most gratifying economy, and, better still, the degree of economy

seemed to increase as the engine grew older; but this struck Mr. Coggeshall as a little too good, so he put in a weir as a check on the pump, with the result of showing that a most excessive slip had developed, due to disordered valves. That weir remains, and an occasional test of the slip of the pumps is regularly made, and the pumps overhauled if the slip is found excessive. And this means of conveniently determining the slip is considered a most valuable auxiliary.

In discussing the report of a committee of the American Society of Mechanical Engineers on a proposed standard method of conducting duty trials of pumping engines, a few months since, I tried to call attention to some of the sources of error possible when using the pump as its own water-meter. Briefly, some of these are:

First.—Backflow of the volume immediately under the lifted valves at end of stroke. In a pump in good order this is small, but if the valve motion should happen to be slow by reason of feeble springs, or inertia of valve, or if valve motion had become sluggish through friction of valve on its stem, this backflow under the valves might be large.

Second.—Although the water cylinders of a pump may be opened and valves found clear, both before and after a trial, a twig or other obstruction may catch under a valve and work out again during a test and leave no sign of the slip it has meanwhile caused.

Third.—Air snifted into the suction purposely to ease the hammer of the pump, or leaking in through some imperceptible and unknown crack may, by causing an imperfect filling of the water cylinders, make the displacement utterly untrustworthy as a measurement of the discharge. I am interested to note in Mr. Holloway's discussion, reference to one more case where something of this kind has happened in practical use, and the record of the pump as its own meter been found untrustworthy.

Fourth.—If a pump cylinder happens to be formed with a cavity from which the air does not get swept out by the water, the alternate compression and expansion of this air may give a fictitious value to the displacement. Even the taking of indicator cards from the water cylinder may not give conclusive evidence as to the absence of air, if speed is high or water hammer excessive. Moreover, the incomplete filling of water cylinders on high-speed pumps, with restricted suction passages, is by no means rare. In the course of some of my experiments made only a month or two ago, we found a Knowle's Duplex 18 x 9 x 12 inches which actually delivered 35 per cent. less water than the displacement called for, due mainly to a leaky suction; and a good Worthington duplex fire-pump 18½ x 9½ x 10 inches was found, which at high speed, from some peculiar cause not yet determined with certainty, delivered 25 per cent. less water than the plunger displacement. In recent careful trials of five other pumps, each of about 1 000 gallons per minute capacity, I have found the total discrepancy less than 4 per cent., and for large, new, slow-moving, pumping engines, I should expect that perhaps

nineteen times out of twenty the total slip would not be more than 3 to 5 per cent.

If we are testing a water-works engine we do not want to rely on something which is merely *probably* correct or *reliable nineteen times out of twenty*, but we desire to have a convenient and handy method which is absolutely certain, and as free from hidden mysteries as is the measuring of a square packing-box with a common two-foot rule. Therefore, I believe that when there is much at stake we should not use the pump as its own meter, but should obtain external evidence by means of either the weir, nozzle, or Venturi meter, and the nearer either of these comes to the standard of simplicity and certainty just mentioned, the better it will be.

I feel very grateful toward Mr. Brush for his proposition to install a large meter nozzle in connection with a large water-works pumping engine, and shall look forward with great interest to its trial.

John C. Trautwine, Jr., Assoc. Am. Soc. C. E.—Mr. Freeman has handled his subject so thoroughly and well in his paper recently read before the Society, and has made out so good a case for his nozzle-meter, that but little room is left for discussion; and all that I propose this evening is to suggest what appears to me to be some fundamental resemblances, and some structural differences, between the nozzle-meter and others that have recently been suggested, and to point out some of their respective advantages and the special applicability of each form.

The nozzle, as used by Mr. Freeman,* and earlier by Mr. Edmund B. Weston,† for the measurement of flowing water, is simply a special case of the general one represented in the Venturi meter developed by Mr. Herschel and in Pitot's tube. The general case is that where the head generating (or destroying) an observed velocity through a pipe or orifice is measured, and the co-efficient c obtained for the formula—

$$v = c\sqrt{2gh}$$

where v is the observed velocity and h the head producing or destroying it. In Pitot's tube, the head destroys the velocity; in the others it generates it. With the nozzle, discharging into air, the pressure at the orifice is simply that of the atmosphere; so that the head h in the formula is simply the height of the water above the orifice, plus the head due to the velocity past the piezometer; while in Pitot's tube and in the Venturi meter, the orifice is submerged, and the head h generating or destroying the velocity is therefore the difference between two heads; one corresponding to that measured in the case of the nozzle, and the other acting directly at the orifice and in the opposite direction. In Pitot's tube this second head is always positive, and must therefore be deducted from the first; in the Venturi it may be either positive, zero or

* *Transactions* Am. Soc. C. E., November, 1889, and the present paper.

† *Transactions* Am. Soc. C. E., November, 1884.

negative, and in the latter case the true head is the arithmetical sum of the two.

All three of these instruments have the great advantage that the co-efficient is very nearly constant, and almost exactly equal to unity; and each apparatus would seem to have its particular advantages for special cases. Pitot's tube is the only one of the three that can be conveniently applied to open channels; the Venturi meter commends itself as a permanent attachment to existing systems of water supply; while the nozzle is well adapted to tests of pumping engines or of fire-hose, has the important merits of simplicity and portability, and (as compared with Pitot's tube) of solidity and consequent freedom from liability to damage. On the other hand, Pitot's tube has a much wider range of applicability. While it is the only meter of the three that is well adapted to the gauging of open streams, it may also be used to good advantage in the spheres of usefulness of the other two. Mr. Freeman has used this remarkable instrument to excellent advantage in his delicate determinations of the velocities in different parts of the cross-section of a hose or nozzle, and M. H. Bazin has recently applied it to perhaps equally delicate and interesting investigations respecting the distribution of the velocities and pressures in the sheet of water passing over a weir.

The development of Pitot's tube from the simple forms proposed by its inventor into that of a most convenient and accurate meter appears to be due to Professor Stillman W. Robinson, M. Am. Soc. C. E., of the Ohio State University, who describes the instrument in detail in *Van Nostrand's Magazine* for March, 1878, pages 255, etc. In the same journal for August, 1886, he describes its application to the gauging of currents of air or of gases.

It would be of great theoretical interest, and perhaps of practical utility, if piezometers were to be established at points close together along the nozzle and play-pipe in experiments like those of Mr. Freeman, so as to give a plotting of the heads, showing their variation between the orifice and the base of the play-pipe. A comparison of Mr. Freeman's earlier experiments, in which the heads were measured at the base of the play-pipe, and the present series, in which it was measured at the base of the nozzle, shows (as might be expected) a much greater uniformity and closer approximation to unity, on the part of the co-efficients, in the latter case; and the question arises, to what extent (if any) can we increase the accuracy of the nozzle meter and its reliability with widely varying forms of nozzle, by placing the piezometer still nearer to the orifice?

Mr. Freeman (by letter).—In reply to Mr. Trautwine and his reference to the work of Professor Robinson in developing the Pitot's tube into a convenient gauging instrument—I would suggest that probably the most extensive practical application of the Pitot's tube ever made in this country or abroad was made by Mr. Hiram F. Mills, Chief

Engineer to the Water Power Company, at Lawrence, Mass., and that his work antedates that of Professor Robinson. Twenty years ago Mr. Mills constructed several of these instruments, some on the general plan of that illustrated in the large work of Darcy and Bazin, but embodying several improvements of much value in practice, and made careful and elaborate determinations of their co-efficients. These instruments were used for many years, and are, perhaps, still in use, for gauging the volume of water supplied for power to some of the large manufacturing corporations at Lawrence, and their indications served as the basis of settlement between seller and buyer in contracts amounting to many thousands of dollars. It takes a good deal of care and skill to manipulate an instrument of this sort so that error shall not creep in, but in the hands of a skillful and experienced observer they will do very fair work even with velocities down to 3 feet per second. Mr. Mills also made many other special researches with apparatus on this principle. No account of these experiments has been published, a fact much regretted by all familiar with their range, accuracy and value.

In reply to Mr. Trautwine's final suggestion, I would reply that I think we now have got the piezometer up as near to the orifice as it is worth while to go—for one leading consideration in the effort to increase the accuracy, is to keep the velocity past the piezometer low; hence it should be back where the area of channel is large. And since our total loss of effect is but 0.005 this comes as near to a co-efficient of unity as we need try to attain.

Professor J. E. Denton.—The matter of tests of pumps is an interesting subject to me, and I will mention one or two facts that have come under my observation. I understand Mr. Freeman's nozzle is an attempt to make a portable substitute for a weir. He does not impeach the accuracy of the weir, nor do I understand that he improves on the accuracy of the weir by this device. It is hard to see how one could do otherwise, when it is understood that the rate based upon the measurement of water in large quantities, for any such calibration as Mr. Freeman has made, could not for a moment sustain the comparison to weir measurements. The nozzle, so far as he has calibrated it, is entirely reliable within the area that he states, which I gather to be about 1 per cent., but I think he has not undertaken yet to calibrate a nozzle larger than 2 inches, and that is equivalent to calibrating 2 000 000 gallons in twenty-four hours. That is a small approach to handling the quantities that are called for in our tests of pumping engines.

Also I understand Mr. Freeman's nozzles have yet to be calibrated under the pulsations of water that are always present with pumps, and that his 2-inch nozzle has only been calibrated under a steady stream, so that we must allow for the accuracy of his work until he has calibrated under a pulsating stream. Admitting the accuracy of this, however, we may say that his 2-inch nozzle is an experiment that we can

accept for such small pumps as he has applied it to, and under two million gallons. Unfortunately, the testing of the water of those pumps is seldom a matter of importance; the tests we want are the larger ones for cities. I doubt if we can accept the statement that a calibration of these nozzles can be substituted for pump or weir measurements. I believe Mr. Freeman has only claimed for the nozzles the advantage of their portability and the quickness and ease with which they can be erected; but when it comes to the test of a large pumping engine, and it is decided that a large quantity of water is to be measured more closely than the use of the pump and indicator cards would give—and I believe their limit is about 2 per cent.—can we trust the nozzle measurement? (It is pretty well established that where the stroke is constant, the difference between the pump and the vertical displacement of the plunger is something inside of 2 per cent.; if the stroke is variable that brings it beyond 2 per cent.) The crank pump, of course, settles its own case. The direct-acting pumps vary in their economy. The crank pumps have practically a positive stroke; I believe they work within a 16th of an inch in 36 inches, so that most experts in pumps are quite sure they can determine whether the pump is safe, within 2 per cent. Granting that this nozzle, on a small scale, has been proved to be as reliable as a weir, still it is yet to be tested in measuring as large quantities as have been measured by a weir, before we can entirely regard it as a substitute.

Mr. Freeman (by letter).—Before the present century, or indeed, probably since the time of Torricelli and of Newton, it has been recognized that the discharge of a simple orifice, nozzles included, could be computed approximately if its diameter and the head acting on its base were known, but the method has heretofore had but little, if any, "professional standing," so to speak. The remarkable capabilities for accurate water measurement possessed by the nozzle came to my notice two or three years ago, and so far as my opportunities have permitted, I have labored to establish the high accuracy of this method of measurement on a firm and scientific foundation. I have not taken the time of the Society by recommending merely a method which promises well, but have already put its working to the practical test up to a fairly good practical size, and have tested it not merely under most favorable conditions, but have, as will be seen by studying the experiments, sought to vary the conditions and to disturb the approaching current—to try nozzles made at different shops—to test them under wide ranges of pressure and of extreme velocities past the piezometer, and finally have used the nozzle as a practical means of gauging, in some pumping engine tests which, while trying to perfect a style of pump for fire duty, I have had occasion to make upon some half a dozen different moderate sized pumps in different cities, set under widely different conditions and some of them running under extreme conditions of speed, water-hammer and violent pulsation. Un-

der all these trials the nozzle has fully met the requirements, and my confidence in its accuracy, has continually increased, and proofs of its great practical convenience have accumulated.

For fifteen years or more I have had considerable experience with weir measurements under various conditions, and have witnessed the use of weirs by others for measuring pump discharges, and have used them myself for this purpose, and while I believe that for the measurement of large volumes (say 3 to 300 cubic feet per second), under low heads (say 1 to 5 feet), the weir will probably always continue the standard method; I feel equal confidence that for measuring moderate volumes (say 100 to 5 000 gallons per minute) under high pressures, 10 feet or upward, and where the jet can be freely discharged into the air, the nozzle will steadily grow in favor and become recognized as fully the equal of the weir in authority, as fast as hydraulic engineers have occasion to put its convenience to the trial. I would not, for an instant, be understood as discrediting weir measurement. Weirs are, however, so expensive and inconvenient to apply that it is becoming too common to do without them and trust to plunger displacement.

One great advantage of the nozzle meter over the weir is its portability and ease of attachment, and that one nozzle can be, if so desired, calibrated once for all and then used at many trials. There is, however, no more real need for calibrating each nozzle than there is for calibrating each weir. That one not experienced may get misled in the use of a weir has often been shown. Eddies in the approach, rounded crest, lack of air under the sheet, improper ratio of length to depth, insufficient depth below the crest and measurement of depth down stream from the point where the surface of water begins to curve, have each got good men into trouble. The recent experiments of Bazin (1886-88) on influence of velocity approach, are also worthy of study in this connection.

(See Annales, Ponts et Chausées, October, 1888, or more conveniently, Mr. Trautwine's abstract and translation in *Engineering News*, December 27th, 1890.)

That the nozzle will prove superior in accuracy to the weir is due: 1st. To there being less uncertainty as to the correction for velocity of approach. 2d. There is with a nozzle of the degree of taper recommended, no uncertainty as to the degree of contraction. 3d. A mercury pressure gauge used in connection with a nozzle measures upon a magnitude so much greater than does a hook gauge over a weir, that the relative precision of the former may readily equal the minute accuracy of the latter. 4th. The discharge of a nozzle varies as the square root of the head. The discharge of a weir varies as the cube of the square root of the head. Therefore, while an error of 2 per cent. in determining the nozzle pressure would cause an error of about 1 per cent. in the quantity gauged by the nozzle, the same error of 2 per cent. in the weir depth would cause an error of about 3 per cent. in the quantity indicated by the weir.

The superiority of the nozzle over the Venturi meter, for cases where the jet can be discharged into the air, comes: 1st. From there being but a single pressure to be measured, instead of having to work on the difference of two pressures. 2d. The play-pipe leading to the nozzle can be large enough to give a slow velocity past the piezometer, and thereby the pressure can be measured with greater accuracy than it can at the swift moving current at the "throat of the Venturi," when any little burr or almost invisible projection at the piezometer, or lack of parallelism of current, might cause a noteworthy error. 3d. The magnitude of the quantity measured, and furnishing a basis for the computation, is greater, and may therefore be measured with a higher degree of relative precision.

The degree of precision mentioned by me as attainable, was intended to be a conservative statement, for I am inclined to believe that the discrepancies which appear in Tables 1, 2 and 3, simply give the measure of the degree of precision of the apparatus available to me for observing time and pressure; and the remarkable series, 55 to 62, taken after my electric chronograph was perfected, incline me to the belief that were it worth the while, one could so perfect the auxiliary apparatus of pressure gauge and time-piece (and by using a large tank, and noting effect of temperature); that if the source of water supply was substantially free from pulsations, the very simple meter nozzle shown in Fig. 7, might measure with certainty to within one-tenth of one per cent. In other words, the discrepancies in our values measured for the co-efficient, do not signify that the co-efficient of discharge really varied to this extent. Moreover, the larger part of the observations given in these tables are with conditions purposely more disturbed than they need be in practice. In some the velocity past the piezometer was purposely made extremely high. In others the current approaching the nozzle was thrown into a somewhat disturbed, eddying condition. In all, except when the chronograph was used, the smallness of the tank, together with discrepancies in the stop-watch caused a lack of regularity not belonging to the nozzle as a meter.

It has been suggested in the discussion that a degree of care not attainable in ordinary practice may be needed to get so high a degree of accuracy in the use of a meter nozzle as within 1 per cent. To this I reply that the only great care needed is care to use a pressure gauge whose accuracy or degree of error is known. Further, in reply to Professor Denton, it may be said that I do, for moderate volumes and high pressures, claim for nozzle measurement an increase in accuracy over weir measurement, but that the margin of uncertainty in a well arranged weir measurement is so small that the reducing of this by ½ or ⅓ is for most practical purposes of less account than are the advantages of compactness, convenience of calibration and portability possessed by the nozzle. The largest nozzle which I have as yet calibrated is 2½ inches

in diameter, instead of 2 inches, as is intimated in Professor Denton's discussion.

The kind and enterprising offer of Mr. Brush will soon give the Society an opportunity to judge of the convenience and accuracy of the nozzle applied to a large water-works engine.

I will return to one point further, which seems to have been lost sight of in the discussion. While questioning if the co-efficients here determined could be used for nozzles of different or larger size, by reminding that if the mere simple theoretical value for the discharge, be used without any co-efficient whatever, the result will still be as accurate as can be attained by any other ordinary form of measurement. In other words, the amount of divergence from the extremely simple, pure mathematical theory, is only one-half of 1 per cent. By the delicate investigation briefly referred to on page 18, and illustrated in Fig. 6, I showed this divergence almost wholly accounted for by the retardation of friction against the walls of the nozzle. Now grant, for argument's sake, that in some other uncalibrated nozzle there was double the skin friction found in any of those which have been calibrated, or grant that the skin friction became trebled by old age or corrosion, the practical error thereby introduced would be probably inside of 1 per cent., or less than is often caused by disturbing circumstances about a weir. And any such roughness as just suggested would be so noticeable that it is hard to conceive of its being tolerated by any competent engineer.

Professor J. Burkitt Webb.—I am sorry that Mr. Freeman is not here, as he might explain a point which I do not understand. Mr. Freeman says, toward the close of his paper: "I also had some apprehensions lest the fact that the sectional area of the conduit opposite the piezometers was slightly greater than a few inches up-stream where obstructed by the 'rifle-blades,' it might throw the piezometer region a little into the condition of a diverging pipe, and thus tend to slightly lower the piezometric column and give an increased value to the co-efficient." As I understand the column, it is a column of water whose height indicates the statical pressure in the piezometer region, so that a lowering of the column means a diminution of the pressure. If, in this region, and by reason of the absence of the "rifle-blades" the section is increased, so that the region is, in fact, the larger portion of a diverging pipe, then, of course, the velocity is diminished, and consequently the pressure must be increased, as also the height of the piezometric column. I have no doubt that the author of the paper will see the (at least apparent) discrepancy, attention being called to it. It is a point, however, which is not understood by many, and it is difficult to convince some objectors of the truth of the theory. If two pipes, alike in all respects, are to be compared, except that in one the velocity decreases by reason of a slight enlargement, while in the other the velocity does not change, then the decrease of velocity must have a cause, and the only

cause assignable is an increase of pressure. The effect of friction and any inclination that the pipes may have, will be substantially the same in both, leaving no other conclusion than that of an increase of pressure where the pipe enlarges. Although we rightly conclude that if the pipe is enlarged it cannot flow full without a decrease in the velocity of the water, yet the enlargement is not the cause of the decrease, but the increase of pressure is. In fact, the only thing that can decrease a velocity is a force in the opposite direction, and this is just what the increase of pressure amounts to, and the water loses its velocity in forcing its way into the region where the increased pressure exists. A steam injector makes use of this principle in forcing water into a boiler where the pressure is greater than the injector, and the water is delivered into the boiler through a diverging tube.

This arrangement of Mr. Freeman's for measuring the flow of water, worked very nicely in a model which I had occasion to make to demonstrate a principle in hydraulics. It is well known that a jet of water issuing from a nozzle exerts a reaction equal to the momentum of the water, but this was disputed, and it was claimed that while a horizontal jet exerted this reaction, an upward or downward jet exerted a different one. The surest way to show the falsity of such a claim is by experiment. Peter Ewart experimented upon a jet of water issuing horizontally from a vessel, and found the law true (see Vol. II, "Memoirs of the Manchester Philosophical Society," or Weisbach's "Mechanics," Vol. I, under "Reaction of Water"), but the experiments with jets in other directions is not so easily made. To weigh the reaction of other jets, say one flowing vertically downward, I constructed an apparatus (see "Franklin Institute Journal," August, 1887, and January, 1888, for cuts and complete discussion), consisting of a tank with an adjustable weir, by means of which the height of the water in it was maintained constant. From the bottom a pipe projected vertically downward, ending in a nozzle of much smaller diameter, from which the downward jet issued. In order to weigh the reaction the pipe was cut a short distance below the tank, and the lower portion supported in position by scales, constructed for the purpose. With the exception, then of the crack, one thirty-second of an inch wide, the pipe was continuous from the tank to the nozzle, and if no water escaped at the crack and no air entered, the desired conditions for weighing the reaction would be obtained. To make sure of this, it is necessary simply to adjust the weir until the head of water in the tank is just sufficient to supply as much water to the hanging pipe as will keep it full, and this is known at once from the fact that if more is supplied it commences to escape from the crack, while if less is furnished air is drawn in and the jet commences to sputter. This crack, therefore, constitutes a simple piezometer for maintaining the pressure at that point in the pipe, equal to the atmospheric pressure, while it also leaves the lower pipe free to

be weighed. The nozzle is first corked up and the pipe weighed full of water, and then it is weighed with the water flowing, the difference being the reaction of the jet issuing from the nozzle minus that of the jet flowing from the tank into the suspended pipe. It seemed very beautiful that the water would jump across the crack, and enter the the lower pipe without loss and without suction, and though the laws of mechanics made me reasonably sure that it should do so, I was surer after seeing it.

Two or three years after making these experiments, we heard of the great things that were to be expected from a new phase of jet propulsion, and I told my students, much to the astonishment of some, that a vessel would go as well with a jet squirting backward into the air as with a submerged jet. One of them got the use of a steam pump, and without telling me until afterward, proceeded to interrogate Nature. He hung up a hose, with a bent nozzle on its lower end, and forced water through it with 100 pounds pressure, measuring the reaction of the jet with a spring balance. In water or out of water the reaction was the same, and when he belabored the jet with a board it didn't stir the hose, nor did holding a board within an inch or two of the end of the nozzle.

The velocity with which a jet issues from a nozzle does not in any degree make it like a solid rod, and such a stream is as incapable of resisting compression as if it had no velocity. It might cut your finger off, but a stream of bullets would do so, or you could get that effect by jerking your finger with the same velocity through still water. In fact, by shooting the same number of rounds of bullets per minute, with the same velocity as the jet backward from a vessel, the same propulsion would be obtained as with the jet.

While the method proposed may be a good way to measure a discharge of water, it would seem advisable in any important experiment not to depend entirely upon any one method if it can be checked by some other, and I would propose as an improvement on the simple nozzle, a nozzle swung up in such a way that the reaction of the issuing jet could be read from a spring balance. This would give the product of the mass issuing per second by the velocity and serve as a check upon the discharge as calculated from the piezometer or the meter.

Mr. Freeman (by letter).—In reply to the first point raised by Professor Webb, viz.: as to my remark near the bottom of page 19, where I was discussing the general question of piezometers, rather than nozzle measurements, and regarding a transitory doubt expressed by me as to a possible slight local disturbance of the piezometric indication, I would say that in a diverging tube with a swift current, the regimen of flow is generally in a kind of unstable condition. The inertia of the swifter moving water tends to drag along the water in the divergence at the sides, and the "suction" on this water tends to produce a vacuum and lower the piezometer, the action being in feeble imitation of the action of the well

known "ejector" pump when driven by a water jet. At one time I feared that this might injuriously affect the reading of that piezometer located under those peculiar conditions. This question has no special bearing on the general subject of gauging by the nozzle.

The main part of Professor Webb's discussion in which he describes his experiments on the reaction of jets, is very interesting to us all, for although theory is clear on these points, it is hard to fully comprehend them and feel sure until the actual trial of such experiments as Professor Webb's. As to measuring the quantity discharged by actually weighing the reaction of a large nozzle, such practical complications would be brought in that I fear no advantage in precision would result.

Some one has raised the point that it requires a delicate computation to interpret the indication of the gauge and give us the information in gallons per minute. If one will construct his nozzle like Fig. 7, so that the velocity past the piezometer shall be slow, the computation becomes one of the most simple of all hydraulic problems. And if once a nozzle is constructed, and its diameter calipered, a table can be easily prepared giving the discharge corresponding to each pound of pressure, so that with the apparatus once set up he can perhaps, all in the short space of twenty seconds, read the pressure gauge, look out the quantity and know the rate of discharge.

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480.

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PROPORTIONAL WATER-METER, SPECIALLY ADAPTED TO INFERENTIALLY MEASURE THE TOTAL DISCHARGE OF NOZZLES.

By JOHN THOMSON, M. Am. Soc. C. E.

WITH DISCUSSION.

The writer is led to more hastily prepare and present this communication than he had otherwise anticipated, in consequence of the recent paper by Mr. John R. Freeman, M. Am. Soc. C. E., entitled "The Nozzle as a Water-Meter," which, presumably, is the outgrowth of his previous very able experiments, the "Hydraulics of Fire Streams," presented to this Society. As I understand it, Mr. Freeman freely offers this device to the profession for their use, more particularly as a means for measuring the discharge of pumping engines during efficiency trials. Therefore I shall herein combine a short discussion in criticism of Mr. Freeman's paper, and submit in like manner and for a similar purpose a device which, it is believed, is better adapted for the contemplated duty.

Some of the objections to the use of Mr. Freeman's system have already been brought up by Mr. Charles B. Brush, M. Am. Soc. C. E. That spring gauges require to be frequently calibrated is well known. It is also a fact, at least in the writer's experience, that ordinary commercial

pressure gauges are quite as liable to lose their accuracy suddenly as to part with their truth gradually and uniformly. There is not to be forgotten what gauge-makers term "sluggishness under tension," the hand indicating after the impact, and continuing to indicate by momentum, thereafter. Then, too, the accurate reading of the hand of the gauge when under considerable tension is in itself an art. But the controlling objection is that the sum of all the uncertainties, whatever they may be, is not applied to the right-hand side of the decimal point, but to the left-hand side thereof. In other words, an unobserved error of small value as to the gauge may be of great value as to the total discharge. Besides, not alone are the aberrations intermittent, but they depend for accuracy upon acquired personal skill or aptitude, while the error may be intermediate to the observations or synchronous therewith. In this connection I may digress to say that Mr. Emil Kuichling, M. Am. Soc. C. E., has recently found that the use of ordinary pressure gauges in piezometric measurements is quite unreliable, and for such purpose not believed to be trustworthy if accurate work is essential. The means which he has substituted, and the experiments in connection therewith, will probably be presented to the Society by him at a later day.

I may also advert to the fact that one of the particular features of Mr. Freeman's experiments in the "Hydraulics of Fire Streams," was his skillfully designed and accurately constructed mercurial gauge; and that the only results which he himself appeared to question were the experiments with the Pitot tube in which a spring gauge was used.

The device which I herewith present is based upon the well-known principle of proportional measurement of volume, and is in fact an inferential proportional water-meter, although as applied to Mr. Freeman's nozzle it appears in the main but a modification of his arrangement, the pressure gauge being dispensed with and simply an ordinary water-meter substituted. In brief, the scheme or principle is, to deflect a small portion of the whole stream, to measure this accurately, and thus inferentially determine the total discharge. The meter is in truth a gauge recording by quantity, but set under conditions which are believed to be conducive to a much higher degree of uniform accuracy, certainty of action and durability, than in the instance of an ordinary gauge, indicating the dynamic pressure in pounds.

In the accompanying drawing, Plate LXI, which is a partial side elevation and longitudinal center section, the conditions just referred to are

illustrated. The pipe, the nozzle, and the piezometer sleeve, are essentially similar to those shown in Fig. 2 of Mr. Freeman's paper, except that the openings *A*, through the pipe leading to the circular chamber, *B*, of the piezometer sleeve are larger, and that a base, *C*, is provided for supporting and securing the apparatus, as to the timber, *E*. The hub, *F*, of the sleeve is adapted to receive an ordinary union, *H*, which connects directly with the inlet chamber of the meter. This connection also serves as a support to the meter. The outlet chamber of the meter is connected by the spud and union, *I*, to a brass outlet pipe which extends forward, parallel with the nozzle, supported by a bracket secured thereto; the right-hand end of the pipe being bent downward and thence outward, until its open end lies approximately in the center of the line of discharge, *K*, of the nozzle.

Secured to the back of the end of the outlet pipe is a plate, *L*, the bottom of which is formed to a conical tube, *M*; its larger opening presented toward the nozzle, and its restricted opening directed to the center of the outlet of the pipe. The outer and upper extension, *N*, immediately above the conical tube, is finished to a knife edge, the more readily to deflect the water past the sides of the outlet pipe. The tie-rod, between the bracket and the plate, is to stiffen the structure.

It will now be seen that the dynamic effect, capable of being produced upon the meter, may be very great, for the reasons that we have, first, direct pressure from the piezometer chamber to the inlet side of the meter; second, practically a direct discharge into the open air; and third, the suction produced upon the interior of the outlet pipe, due to the water flowing past the outer edge thereof at *O*, and the additional suction due to the water which directly impinges upon and flows through the conical tube, acting in accordance with the principle of the injector to cause secondary flow by induction.

We thus have the necessary conditions—namely, ample difference of pressure between the inlet and the outlet of the meter—to produce in it the most uniformly accurate results as to its operation; which will be when the meter is worked at a fairly uniform rate of discharge and with a maximum duty not exceeding, say, three-fourths of the usual advertised capacities. Such conditions are readily obtainable under the requirements here contemplated, in that the fluctuations of discharge during the ordinary duty test of such engines are comparatively limited. The calibration of the meter may be readily obtained by delivering, at

different rates of flow, into a receptacle of known capacity, or more accurately by weighing the water; when, if the indications made by the register of the meter are uniform, a co-efficient is established to obtain from the fixed automatic record of the meter the total quantity of future deliveries. The function of the stop-cock shown in circuit with the outlet pipe is to assist in such calibration. Thus, with the cock closed the water may be run to full capacity of the nozzle before discharging to the trial tank; and when all is ready, the nozzle may be deflected and the cock opened practically simultaneously. So, too, the flow through the meter may be very quickly stopped without danger of injury from water shock.

In testing meters by short runs it may be well to note that, whenever practicable, a volume should be delivered sufficient to insure at least one complete revolution of the primary hand of the register. This is for the reason that the register dials of commerce are not accurately graduated in the subdivisions of the unit of measurement.

In a calibration of this kind, the first step would be to ascertain and fix the maximum operation of the meter. The limitation may be readily accomplished in the ordinary manner, that is by applying a throttling diaphragm between the outlet spud of the meter and the union. It may also be noted that when the meter is once calibrated the register may be adapted to indicate the total quantity and not the proportional; but as this would be a matter of convenience only, it might not be warranted in practice. The employment of the separate induction tube *M* is not regarded as an element absolutely necessary, and the apparatus might be simplified to the extent of its omission.

As to the type of meter to be employed, the writer does not hesitate to say that under the conditions herein set forth, the light, compact, positive displacement rotary meters would be the preferable; not only because of the convenience in manipulating, but in that they usually have a higher discharging capacity with the least resistance to flow. There are a number of such styles of meters in the market which might be readily adapted to the conditions and duty here indicated and described.

As regards the size of meter, I can think of no advantage likely to come from the use of instruments larger than the ⅝ inch, ¾ inch and 1 inch sizes, to be worked to maximum capacities of say 10, 20 and 40 gallons, respectively, a minute; for the reason that the meter employed

must operate with equal accuracy, whether its proportional value, or coefficient, be as 1 to 10, or to 50, or to 100. The determining factor in the selection of the size, however, might necessarily depend upon the facilities for calibrating.

In conclusion, I deem it but proper to state that although, for lack of facilities, I have not tried this device in actual practice, it is yet not entirely a theoretical deduction, but the outgrowth of the actual application of its essential principles, in the design of proportional water-meters for use under the much more difficult requirements of regular service. And irrespective of the fact that there are here shown some features subject to patent monopoly, I would say that any member of this Society is free to use the same for the purpose primarily specified in this paper.

DISCUSSION.

Mr. Lewis H. Nash.—In some experiments I have made, I was surprised at the accuracy which could be maintained. In my experiments the meter was on a small scale. The meter I speak of was of 2-inch capacity. The attempt was not only to obtain an accurate result with a large flow of water, but to get a meter which, under all conditions of flow, would give a substantially accurate result. In order to obtain a constant relation between two streams of water; you must have some way of obtaining two resistances which will bear a constant relation to each other; so that if you measure one stream you can, with accuracy, determine the other. I found that the difficulties in relation to measuring came principally from the resistance of the measuring devices themselves, that is to say, the meter. You will find that all meters have a certain amount of resistance due to their construction, such as velocity of the moving parts and friction in the pipes which connect the meter; whereas the resistance of the main stream would be a comparatively simple thing. So, if you undertake to force the meter in any sense, the deductions you obtain will be void. I used in my experiments a series of holes of the same size, one of which passed the metered stream and the others of which passed the main stream.

In order to secure accuracy it is only necessary that the resistance to flow through the meter be kept proportionally small, and when you have a difference of pressure driving the water through the orifices, of 2 or 3 pounds, the resistance of the meter is negligible if it is not overforced, and in this case the flow of water through the two channels will be substantially proportional to the rate of discharge of the several noz-

PLATE LXII
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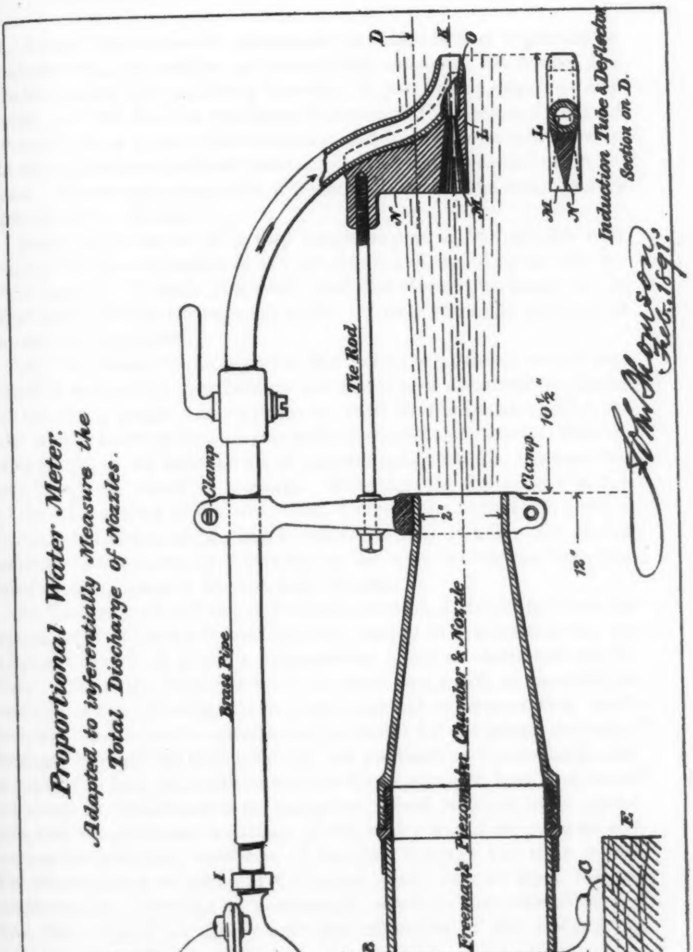
PROPORTIONAL WATER METER.



Proportional Water Meter

zles. I found that when the pressure at the orifice is 2 or 3 pounds to the square inch, the results are remarkably constant. Of course, the limits to accuracy are something like this: if you force a large quantity through, you will find the resistance increases in the measured channel more rapidly than in the other channels; and if you are measuring very light streams the proportional resistance in the meter channel again increases. This may be due to the stiffness of the water, the water prefers to take the other channel.

I should say it would be a very simple matter if two nozzles were used, one to index the meter in any condition, and the other to take the stream directly. I think you would find the stream discharge was in perfect proportion to the capacity of the nozzles, when the pressure at the nozzle is considered.

Prof. J. E. Denton.—In regard to Mr. Thomson's paper, I am not sure whether it is intended to substitute the meter as a more reliable device than the spring gauge. In one case we have to determine whether the meter is more liable to keep its rate under the shocks of the water than the spring gauge, or we have the set of experiments that Mr. Freeman has made, using the meter as a gauge. Knowing Mr. Thomson's meter, and the other meters of the same class, I have no doubt that it is quite as reliable as the spring gauge, but I cannot see that it helps the general confidence in the meter as compared to the weir, which has been confirmed so many times in the last half century.

Mr. Thomson.—Replying to Professor Denton, I would say that the application of the meter is here intended simply as a substitute for the gauge as a record; it is not a proportional meter as described by Mr. Nash. I think Mr. Nash has made the point very nicely in regard to the condition of flow. I regard it as quite material to success that under the conditions shown in substituting the meter for the gauge, the rate of discharge through the nozzle should not fluctuate very materially, and, as a matter of fact, in conditions of the kind spoken of here, this would not occur; nevertheless it is my judgment, based on some little experience, that the frictional resistance of the meter would be quite as constant as the frictional resistance of the flow through the main nozzle. The standardizing or rating of the meter would depend upon the test measurements. It would be necessary to measure the entire discharge from the nozzle, then simply note the indication on the meter which gives the relation between the two.

The Chair (Mr. A. Ftley, Vice-President).—Do I understand that in your apparatus you take the area of the pipe which conducts water to the meter and compare it to the area of the main pipe?

Mr. Thomson.—No, I have nothing to do with that. I take a pipe, say 3 or 4 inches in diameter, and attach any size of meter, then discharge through the nozzle a certain quantity of water into a tank, and find for a certain quantity of water delivered at different rates of dis-

charge, say possibly at 50 cubic feet a minute or 40 or 60 feet a minute, that for a like quantity of water discharged into the tank there is the same indication on the meter every time. For say 50 feet you find 1 cubic foot on the meter, although the fluctuation in the rate of discharge may vary from 50 to 25 cubic feet a minute; you establish the constant ratio of the indication on the meter to the total quantity of discharge. It is simply the attachment of the meter, the measurement of the total quantity of the water, and then finding out what the relation is by the register of the meter.

The CHAIR.—You have to make a previous experiment by which to find out the full amount of water that is furnished by the whole apparatus.

Mr. THOMSON.—Yes, that is it. I have made many experiments on apparatus that are analogous to this, and at ranges of discharge from 20 to 55 cubic feet a minute there is no variation; hence it is not essential that the flow should not fluctuate. Take a ⅝-inch meter. I should say it should be operated so as not to discharge more than, say, 1¾ cubic feet a minute. There are a number of commercial meters now in the market which may range in discharge from 1½ down to ¼ cubic feet a minute, and yet register with practical accuracy.

The CHAIR.—This method would require the constant use of a nozzle of the same pattern, or it would require previous different calibrations. It will require a long series of experiments under different pressures and with nozzles of different sizes, in order to be able to answer any call that anybody should make for measuring water.

Mr. THOMSON.—In all apparatus of this kind—we will say a 3-inch diameter of nozzle with any size of meter applied to it, you cannot take that meter and apply it to another 3-inch nozzle with any certainty of results. Each apparatus by itself would be required to be calibrated by itself.

The CHAIR.—Certainly. I mean to say that if you are to be ready to answer anybody's call to gauge water with that particular nozzle, you would have to try the whole apparatus under various heads in order to know just what it does; it might happen in practice that you have such conditions to meet, and if you are called upon to measure quantities of water different from what that nozzle could measure, each should be tried in the same manner so that you could be sure of your results.

Mr. THOMSON.—That is it exactly. I am simply suggesting a substitute for Mr. Freeman's gauge, a meter which in my opinion, is more accurate than the spring gauge. I am not proposing it as a commercial meter; but when calibrated under conditions such as set forth by Mr. Freeman it is more accurate, in my judgment.

The CHAIR.—Then I was mistaken, because I believe Mr. Freeman suggests his apparatus as a commercial apparatus; you do not put yours on the same basis.

Mr. THOMSON.—No, and I do not understand that he does; I did not suppose that Mr. Freeman had any such intention.

J. C. TRAUTWINE, Jr., Assoc. Am. Soc. C. E.—I understand Mr. Freeman as claiming that any nozzle of ordinary commercial form and in good condition will give without calibration fairly approximate results with the formula:

$$v = 0.995 \sqrt{2\,gh}$$

in which h is the pressure shown by a piezometer placed at the base of the nozzle.

The proportional or inferential meter suggested by Mr. Thomson in his paper now presented, while it also depends upon the pressure at a point in the supply pipe a short distance back from the orifice, requires no measurement of that pressure, or of the velocity through the orifice, but depends simply upon the establishment of the ratio between the total quantity discharged and the readings of an ordinary water meter through which is passed a small portion of that quantity. It is thus seen to be radically different in principle from the meters referred to in my discussion of Mr. Freeman's paper, and it is much to be desired that Mr. Thomson will put his suggestion to the test of practice and communicate the results in a later paper.

The CHAIR.—It is my recollection that Mr. Freeman went so far as to state that he could guarantee those results within a small percentage, providing the angle is within certain limits and providing there are no irregularities in the nozzle.

Mr. TRAUTWINE.—My recollection was that any ordinary nozzle could give satisfactory results without calibrating.

Professor DENTON.—I think Mr. Freeman's nozzle cannot be used quite so freely as that; as I understand Mr. Freeman, there are some very delicate calculations before you can use it; I do not think this apparatus can be used any more often than the weir can.

Mr. J. F. HOLLOWAY.—I think the point is well taken in regard to the nozzle method of measuring the flow of water. It seems to me rather questionable whether accurate measurements can be made by frictional resistance alone. Accurate measurements I think must take place by the displacement of something solid. The method which Mr. Thomson presents here is simple, as he says; being a device by which he hopes to obtain a better result than by the use of the spring gauge, but the whole apparatus depends largely on the condition of the outlet nozzle, and I think that it would be quite unsafe for anybody to assume that any nozzle of a given dimension would invariably produce certain results. We know, and there are some here who may remember, that in the old hand fire engines great care was exercised in having the interior of the nozzle not only constructed on the best lines but kept in the very best order, and that the care of the "nozzle" was always a sacred trust; it was always kept most accurately as to its dimensions and its bore

polished to the highest degree; this was regarded as an essential feature of the old hand machine. Now a nozzle may be damaged in some way by corrosion or otherwise, or it may be by the action of dirty water passing through it, and if so, it would upset the nicety of adjustment which is so essential to accuracy of results.

As to measuring the amount of water discharged by means of a nozzle, it strikes me as an excellent way of doing it, either as carried out by Mr. Freeman or with the variations proposed by Mr. Thomson. The comparisons made by Mr. Trautwine are interesting, and prompt me to add one more. To calculate the current (which, we may suppose, is too large to be measured directly) flowing in any electrical conductor, we need to know simply the fall of potential, or electro-motive force between two points and the resistance of the wire, and if the latter remains the same, the current varies with the fall of potential. But we may avoid a direct measurement of the difference of potential by "shunting" off a portion of the current, sufficiently small to be measured, and causing it to flow in a parallel wire between the same two points; then the current in the main wire will bear the same ratio to the current in the shunt as the resistance of the shunt bears to that of the main wire.

Mr. Freeman's plan corresponds to the first method; he measures the difference of pressure which causes the flow through a nozzle when resistance has been previously determined, and from this difference he calculates the discharge. Mr. Thomson proposes to follow the second method and shunt off a portion of the flow, measuring it by means of a meter and deducing the whole discharge from the registration of the meter. The only difference between Mr. Thomson's plan and that used by electricians is that the resistance of a homogeneous wire of uniform section is a simple thing to measure, and therefore, the calculation of the current is correspondingly simple, while, in the case of the water flowing through a nozzle, and through the meter and its connections, the channels are neither of uniform section, like the wire, nor homogeneous, if we may apply that term to a channel in which the resistance varies inversely as the section; consequently the calculation of the flow cannot be made on the principle, if the ratio of the resistances and the whole apparatus with meter attached has to be calibrated, so as to find by experiment the ratio between the discharge through the nozzle and that through the meter.

Professor WEBB.—In reference to the last paragraphs of Mr. Freeman's paper, where the nozzle is supposed to discharge "into the pump well," does any one know what the effect of the entangled air in the water is? Such air is in the shape of bubbles, so small as to be practically invisible, and it seems to me it would increase the apparent duty of the engines. Would it not do so? It would be very difficult to tell how much air there was and to make a correction for it.

Mr. HOLLOWAY.—It is much easier pumping air than water. There

was some discussion at one time as to the quantity of water that was pumped in a trial of engines in a western city, in which the superintendent of water-works claimed that an immense amount of water had apparently been pumped by the engines as measured by plunger displacement, but which, in fact, was far less, owing to the admission of air under the suction valves. I think, very likely, the presence of air in this case would greatly reduce the flow of water through the measuring nozzle.

Mr. Fteley.—Was the fire put out a little quicker in the case you speak of?

Mr. Holloway.—I think its only effect was in the showing of the annual report of the superintendent.

Adjourned.

Mr. Trautwine (by letter).—I had no intention of claiming, on Mr. Freeman's behalf, that a nozzle of any shape and in any condition will give accurate results, but merely that any ordinary shape in good condition would give usefully approximate results, and that this is within Mr. Freeman's claim appears, I think, from a reference to the answers he makes to questions 3 and 4 on the 16th page of his paper.

I did, however, omit to refer to the correction for velocity of approach (I do not remember any other) required by the piezometer readings. But for this correction * Mr. Freeman gives a formula (page 15), so that no calibration is required on this account; and the point which I wished to make remains established, viz.: That whereas Mr. Thomson's differential nozzle meter absolutely requires a special calibration, at least for every change in the relative capacities of the nozzle and the meter, Mr. Freeman's pressure nozzle meter, under the reasonable conditions specified in his paper, would ordinarily require no calibration.

John R. Freeman, M. Am. Soc. C. E. (by letter).—In discussing the interesting contribution of Mr. Thomson to the study of nozzle measurement, I would first state that my ideas as to the special value of the nozzle for a means of gauging, apply almost solely to its use for tests or measurements of comparatively short duration and not for integrating a long continued flow. I do not regard the nozzle as possessing any special merit for this latter purpose, except it be observed by some persons at frequent intervals. Precisely as with the weir, the nozzle is an excellent device for exhibiting the rate of flow, and like it, is not specially adapted for use in connection with mechanical devices belonging to the present state of the art to give a record of the aggregate flow, where the greatest accuracy is desired. For such accurate measurements as in an engine test, where the nozzle meter might naturally be used, I regard the attachment proposed by Mr. Thompson as involving a sacrifice of simplicity and as introducing possibilities for error.

* Within the range of Mr. Freeman's experiments the correction in the head did not reach 25 per cent.; corresponding to a correction in the co-efficient of about 10 per cent.

Thus: *First.*—With regard to the orifices *A*, if a current is steadily drawn through them, as proposed by Mr. Thomson, they at once lose their distinctive character as piezometers, as the term is ordinarily understood.

Second.—On account of this current, a twig or leaf or small obstruction would be more liable to get caught in the piezometer orifice, and thus deflect the current toward or away from it.

Third.—By the addition of the meter a serious complication is added, for the proportion of whole current which will pass through it depends, as with electrical currents, on the resistance of this shunt. Any little thing such as wear, corrosion, or a grain of sand which would in nowise seriously interfere with the accuracy of this meter itself and not affect the accuracy of a meter under ordinary conditions, might seriously affect the ratio of the two currents, by increasing the frictional resistance to flow through the meter.

Fourth.—Any corrosion, obstruction or change of resistance within the brass pipe, stop-cock, or deflector, would change the proportion of the measured current to the main current as just previously described.

I fully appreciate the desirability of the good thing toward which Mr. Thomson is striving, and for one am grateful for his effort to help us get there, but for the extremely rigorous conditions of such tests as a duty trial of a pumping engine, believe that I can see possibilities for trouble which would destroy any feeling as to certainty of accuracy, even though an instrument of the kind proposed should make a good showing during its trial and calibration.

Almost the sole feature as to which serious question may be raised in the use of the nozzle as a meter or on which doubt may be thrown, is that as to the accuracy of the gauge. I have had quite a good deal of experience with Bourdon gauges of various makes, sizes and kinds, and know that even the best test gauges do sometimes get strangely deranged. The question of securing accuracy by a Bourdon gauge is by no means hopeless, as the many refined investigations continually being made in steam engineering by their use plainly shows. In the most rigid and exacting duty trial, the Bourdon gauge on the nozzle will measure pressure just as certainly and accurately as will the Bourdon gauge on the steam pipe, and indeed with greater accuracy, for as previously pointed out in the paper, one per cent. error in the gauge leads to but one-half of one per cent. error in the computed flow. In any important work in either steam or water, all Bourdon gauges should be often tested. Mere comparison with a standard gauge is not fully satisfactory.

In the paper I referred to the Crosby gauge tester. This is a very simple device made by the Crosby Gauge and Valve Company of Boston, and listed at $50. It consists simply of a piston very accurately finished to ¼ square inch area, and surmounted by a scale-pan on which weights

can be placed. Its piston cylinder is filled with oil and communicates by a pipe to a screw coupling to which the gauge under question may be attached. The great and peculiar merit of the device is found in the way the effect of piston friction is annihilated by simply spinning the scale-pan and piston while the load is on. This rapid rotary motion lets the piston seek its true level free of the effect of friction. The whole device occupies less than a cubic foot of space, and is tenfold quicker of manipulation in gauge testing than a mercury column, and is superior to the ordinary mercury column in accuracy. It can be calibrated absolutely at any time by merely a ten-pound weight and a micrometer caliper, with no disturbing question as to specific gravity or purity of the mercury or as to cistern level or capillarity, and without such fears as that the scale of the mercury column was not properly graduated. I have had one of these in practical use for several months and am greatly pleased with its performance, and would recommend one of them as a part of the outfit of any expert who conducts an important engine test.

Mr. Thomson.—Regarding the suggestion of Professor Webb to infer the quantity of discharge, first, by measuring the reaction of the nozzle; or, second, by measuring the velocity of the flow in a tube, I would suggest that either of these schemes would involve the same difficulty which Mr. Freeman and myself have attempted to obviate, in that the discharge of the nozzle being variable in its rate it would therefore become necessary to provide means whereby to record the extent and duration of the variation in the reactions of the nozzle, or the changes and the duration thereof in the velocity of flow. I do not pretend to say that this might not be satisfactorily accomplished, but I am of the opinion that either of these plans present quite as grave difficulties and would probably be open to the same criticisms as those already presented.

In fine, the fundamental principle here involved is precisely the same in each and all of the several modifications, and may be briefly presented as follows: 1st. It is proposed to measure the volume of water discharged from the nozzle. 2d. The mode of ascertaining the volume is not by measurement of the whole, but by inference. 3d. The rate of discharge from the nozzle is not constant, but variable.

Now, the means suggested for performing the functions of inference are: 1st. By Mr. Freeman, a gauge capable of indicating (but not recording) the changes of pressure due to the variation in rate of discharge. 2d. By the writer, a meter adapted to actually measure and record a portion of the total discharge. 3d. By Professor Webb, means (not specified) for measuring and recording the extent and duration of the reactions of the nozzle, due to changes in the velocity of flow. 4th. Also by Professor Webb, means (not specified) for constantly measuring and recording the variable velocity of flow in a tube.

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THE FALSE ELLIPSE REDUCED BY EQUATIONS OF CONDITION.

By ARTHUR S. C. WÜRTELE, M. Am. Soc. C. E.

Elliptical curves are often used in engineering structures, but from the mechanical difficulties in striking such curves, they are generally replaced by curves struck with arcs of circles from two or more radii at different centers, according to formulas given in engineering works generally limited to 3 and 5 center curves, beyond which there are few rules or instructions how to proceed.

In true elliptical curves the radii vary constantly, according to certain laws, and the curve may be represented by the general equation of condition $a^2 y^2 + b^2 x^2 = r^2 b^2$, from which all the properties of the curve may be deduced. Curves struck with radii, varying only at fixed points, cannot be so simply represented, though equations of condition can be found by means of which a curve of any number of centers may be struck. The best curve described with arcs of circles, will be the one most nearly approaching to a true ellipse. Now the curvature of an ellipse at any point, is the radius of the osculatory circle at that point; and the osculatory circle, is that which has contact of the second order, with an ellipse, or of which the first and second differential co-efficients

of circle and ellipse equations, are equal. The differential equation for radius of osculatory circle is $R^2 = \frac{\left\{1 + \left(\frac{d\,y}{d\,x}\right)^2\right\}^3}{\left(\frac{d^2\,y}{d\,x^2}\right)^2}$ which gives for maximum and minimum radii of ellipse at ends of diameters;—$r_{max} = \frac{a^2}{b}$, and $r_{min} = \frac{b^2}{a}$, which should be the limiting radii of first and last curve in a multicenter curve: that is, the longest radius should not exceed $\frac{a^2}{b}$, and the shortest radius should not be less than $\frac{b^2}{a}$, in a curve approximating to a true ellipse.

The general equations of condition for a curve of any number of centers, may be stated as follows for one quadrant:

Let r_{1} r_{11} r_{111} .... r_n be the radii, commencing with the greatest.
" A_1 A_{11} A_{111} .... A_n be the angles used with radii.
" d_1 d_{11} d_{111} .... d_{n-1} be the differences of radii.
" a = half span, b = rise.

The following general equations must be fulfilled:

I. d_1 (sin. A_1 + cos. A_1) + d_{11} [sin. ($A_1 + A_{11}$) + cos. ($A_1 + A_{11}$)] .... + d_{n-1} [sin ($A_1 + A_{11}$ .... + A_{n-1}) + cos. ($A_1 + A_{11}$ .... + A_{n-1})] $-(d_1 + d_{11} + \ldots\ d_{n-1}) = a - b$.

II. d_1 cos. A_1 + d_{11} cos. ($A_1 + A_{11}$) + ... d_{n-1} cos. ($A_1 + A_{11} + \ldots$ A_{n-1}) $+ b = r_1$.

III. d_1 sin. A_1 + d_{11} sin ($A_1 + A_{11}$) + ... d_{n-1} sin. ($A_1 + A_{11} + \ldots$ A_{n-1}) $- a = - r_n$.

Now, by assuming, in the first place, the angles varying as may be desired, and filling in equation I, d will be obtained (either directly or varying in any given ratio, by substituting ρd for d_{11} and so on), so long as the equations of condition are fulfilled and result in positive numbers. Then d from equation I is replaced in equations II and III, and longest and shortest radii are obtained.

Example.

1. Let $a = 50$, $b = 30$.

Required an arch of seven centers (four arcs in each quadrant) with differences of radii equal, and angles $A_1 = 25°$, $A_{11} = 25°$, $A_{111} = 20°$, $A_{1111} = 20°$, $d_1 = d_{11} = d_{111}$.

From I. $d = (a - b) \div \left\{ \begin{array}{l} \sin.\ A_{\text{I}} + \sin.\ (A_{\text{I}} + A_{\text{II}}) + \sin.\ (A_{\text{I}} + \\ \quad A_{\text{II}} + A_{\text{III}}) + \cos.\ A_{\text{I}} + \cos.\ (A_{\text{I}} + \\ \quad A_{\text{II}}) + \cos.\ (A_{\text{I}} + A_{\text{II}} + A_{\text{III}}) - 3 \ldots \end{array} \right\} = 19.61$

From II. $r_{\text{I}} = b + d\ [\cos.\ A_{\text{I}} + (\cos.\ A_{\text{I}} + A_{\text{II}}) + \cos.\ (A_{\text{I}} + A_{\text{II}} + A_{\text{III}})] \ldots\ldots = 67.08$

From III. $r_{\text{IV}} = a - d\ [\sin.\ A_{\text{I}} + \sin.\ (A_{\text{I}} + A_{\text{II}}) + \sin.\ (A_{\text{I}} + A_{\text{II}} + A_{\text{III}})] \ldots\ldots = 8.26$

These results, being all positive, show the curve to be possible, and can be constructed as below:

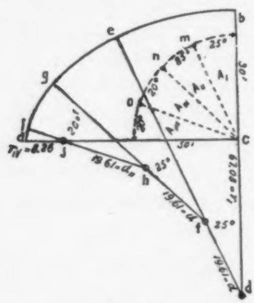


Fig. 1.

General Process of Construction.

Construct rise *bc* at right angles to half span *ac*, and produce to *d*, making *bd* equal to longest radius *R;* set off the given angles by lines *cm, cn, co,* from center, draw *de* parallel to *cm;* set off *df* equal to *d;* draw *fg* parallel to *cn;* set off *fh* equal to *d*, and so on, and if construction is correct *ja* will be equal to r_{IV}.

2. Taking the same example and introducing the condition that the differences of radii vary in geometrical ratio we have

$$a = 50,\ b = 30,\ A_{\text{I}} = 25^\circ,\ A_{\text{II}} = 25^\circ,\ A_{\text{III}} = 20^\circ,\ A_{\text{IV}} = 20^\circ.$$

$$d_{\text{II}} = \rho d_{\text{I}},\ d_{\text{III}} = \rho^2 d. \quad \text{Let } \rho = 0.5.$$

From I. $d_{\text{I}} = (a - b) \div \left\{ \begin{array}{l} \sin A_{\text{I}} + \cos.\ A_{\text{I}} + \rho\ [\sin.\ (A_{\text{I}} + A_{\text{II}}) + \\ \quad \cos.\ (A_{\text{I}} + A_{\text{II}})] + \rho^2\ [\sin.\ (A_{\text{I}} + A_{\text{II}} \\ \quad + A_{\text{III}}) + \cos.\ (A_{\text{I}} + A_{\text{II}} + A_{\text{III}})] - \\ \quad (1 + \rho + \rho^2) \ldots\ldots \end{array} \right\} = 33.13$

$d_{\text{II}} = d_{\text{I}} \times \rho \ldots\ldots = 16.56$

$d_{\text{III}} = d_{\text{I}} \times \rho^2 \ldots\ldots = 8.28$

From II. $r_{\rm I} = d\,[\cos.\ A_{\rm I} + \rho \cos.\ (A_{\rm I} + A_{\rm II}) + \rho^2 \cos.\ (A^1 + A_{\rm II} + A_{\rm III})]$ .... $= 73.50$

From III. $r_{\rm IV} = d\,[\sin.\ A_{\rm I} + \rho \sin.\ (A_{\rm I} + A_{\rm II}) + \rho^2 \sin.\ (A_{\rm I} + A_{\rm II} + A_{\rm III})]$ .... $= 15.53$

These results, being all positive, show the curve to be possible, and it can be constructed as below:

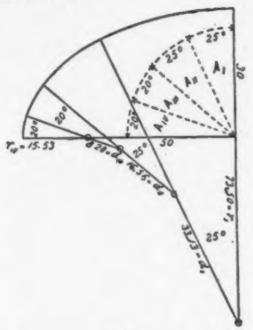


Fig. 2.

3. For three center curves the equations of condition reduce to a simple form:

From I...................... $d = \dfrac{a - b}{\sin.\ A_{\rm I} + \cos.\ A_{\rm I} - 1}$

From II..................... $r_{\rm I} = d \cos.\ A_{\rm I} + b$

From III.................... $r_2 = a - d\,(\sin A_{\rm I})$.

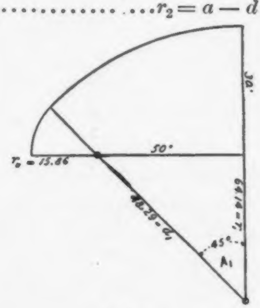


Fig. 3.

From which by substitution of d:

$$r_{\rm I} = \frac{a \cos.\ A_{\rm I} + b\,(\sin.\ A_{\rm I} - 1)}{\sin.\ A_{\rm I} + \cos.\ A_{\rm I} - 1}$$

$$r_{\rm II} = \frac{b \sin.\ A_{\rm I} + a\,(\cos.\ A_{\rm I} - 1)}{\sin.\ A_{\rm I} + \cos.\ A_{\rm I} - 1},$$ and when $\dfrac{r_{\rm I}}{r_{\rm II}}$ is called ρ

$$\rho = \frac{r_1}{r_{11}} = \frac{a \cos. A_1 + b \sin. A_1 - b}{a \cos. A_1 + b \sin. A_1 - a}$$

EXAMPLE.

Let $a = 50'$, $b = 30'$, $A_1 = 45°$.

From I .... $d = \dfrac{a - b}{2 \sin. A_1 - 1} = 48.29$

From II .... $r_1 = b + d \cos. A_1 = 64.14$

From III .... $r_{11} = a - d \sin. A^1 = 15.86$

which may be constructed as Fig. 3.

4. Considering the ratio between the radii

$\rho = \dfrac{r_1}{r_{11}} = \dfrac{a \cos. A_1 + b \sin. A_1 - b}{a \cos. A_1 + b \cos. A_1 - a}$, and taking $b = ya$, we have $\rho = \dfrac{\cos. A_1 + y \sin. A_1 - y}{\cos. A_1 + y \sin. A_1 - 1}$, from which by inspection and differentiation,* it is found that ρ will be a minimum when $y = \tan. A_1$. It will become infinite when $\cos. A_1 + y \sin. A_1 = 1$, or $y = \cos. A_1$, because the denominator then becomes 1 — 1 or 0. The second radius then becomes 0. Since to have ρ infinite in equation, $\rho = \dfrac{r_1}{r_{11}}$, r_{11} must reduce to 0. In this latter case, the curve becomes a circular segment, and A_1 a maximum.

Let y=0.9, then ρ is a min. when $A_1 = 41°59'$ and a max. when A_1 is $83°58'$

.8,	"	"	38.40	"	"	77.20
.7,	"	"	35.00	"	"	70.00
.6,	"	"	30.58	"	"	61.56
.5,	"	"	26.34	"	"	53.08
.4,	"	"	21.48	"	"	43.36
.3,	"	"	16.42	"	"	33.24
.2,	"	"	11.19	"	"	22.38
.1,	"	"	5.43	"	"	11.26

The most regular three-center curve is formed with ρ a minimum.

Let $a = 50$, $b = 30$, y is 0.6 and A is 30° 58′ by equations for three-

---

* Differential co-efficient of $\rho = \dfrac{\cos. A_1 + y \sin. A_1 - y}{\cos. A_1 + y \sin. A_1 - 1}$

$$\frac{d\rho}{dA} = \frac{(\cos. A + y \sin. A - 1)(-\sin. A + y \cos. A)^1 - (\cos. A_1 + y \sin. A_1 - y)(-\sin. A_1 + \cos. A)}{(\cos. A_1 + y \sin. A_1 - 1)^2}$$

made equal to 0 for minimum, denominator cancels out and numerator is reduced by factoring:

$$(y - 1)(-\sin. A_1 + y \cos. A_1) = 0$$

Therefore $$-\sin. A_1 + y \cos. A_1 = 0$$

$$y \cos. A_1 = \sin. A_1$$

$$y = \frac{\sin. A_1}{\cos. A_1} = \tan. A_1 \text{ where } \rho \text{ is a minimum.}$$

center curve. Then $d = 53.76$, $r_1 = 76.10$, $r_{11} = 22.30$. The curve may be constructed as follows:

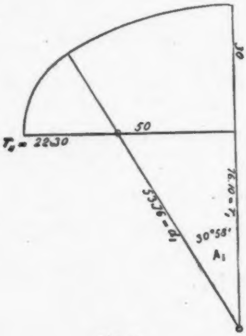


Fig. 4.

6. For a five-center curve the equations of condition may be reduced as follows :

I. d_1 (sin. A_1 + cos. A_1) + d_{11} [sin. $(A_1 + A_{11})$ + cos. $(A_1 + A_{11})$] $-(d_1 + d_{11}) = a - b$.

II. d_1 cos. $A_1 + d_{11}$ cos. $(A_1 + A_{11}) + b = r_1$.

III. d_1 sin. $A_1 + d_{11}$ sin. $(A_1 + A_{11}) - u = r_{111}$.

d_1 will have some ratio to d_{11}. Let $d_{11} = \rho d_1$.

Then $d_1 = (a - b) \div \left\{ \begin{array}{l} \text{sin. } A_1 + \text{cos. } A_1 - 1 + \rho\ [\text{sin. } (A_1 + A_{11}) + \\ \quad \text{cos. } (A_1 + A_{11}) - 1]. \end{array} \right.$

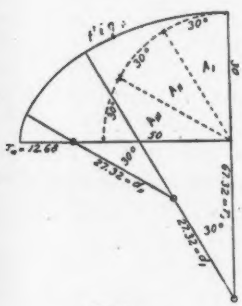


Fig. 5.

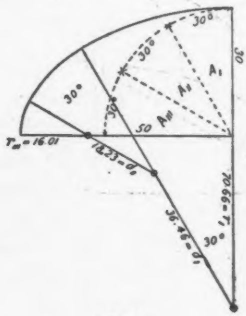


Fig. 6.

$r_1 = d$ [cos. $A_1 + \rho$ cos. $(A_1 + A_{11})$] $+ b$.

$r_{111} = a - d$ [sin. $A_1 + \rho$ sin. $(A_1 + A_{11})$].

Let $a = 50$, $b = 30$. $A_1 = A_{11} = A_{.11} = 30°$, and let $\rho = 1$.

From which $d = 27.32$, $r_1 = 67.32$, $r_{111} = 12.68$.

Let $\rho = 0.5$. Then $d = 36.46$, $d_{11} = 18.23$, $r_1 = 70.66$, $r_{111} = 16.01$.

Special Constructions.

Given AC half span and BC rise at right angles.—Three-centered curves.

Construction No. 1. Let $r = \frac{BC^2}{AC}$ = least radius AG.

Take $CD = CB$, draw AB.

Make DE parallel to AB.

Take $BF = CE$.

Take $AG = CE$. Draw GF.

Draw IH perpendicular to GF bisected in I.

Then H and G are centers required.

With radius HB at center H draw arc BJ, then with radius GJ at center G, draw arc JA, completing the half curve.

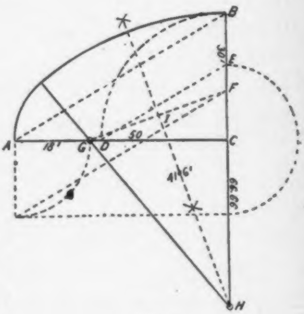


Fig. 7.

Construction No. 2. From Mahan. Take $\frac{R}{r}$ minimum.

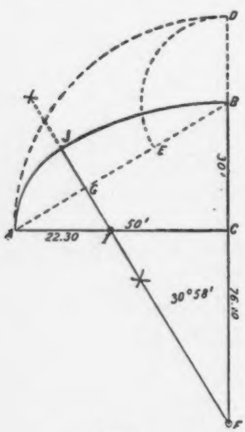


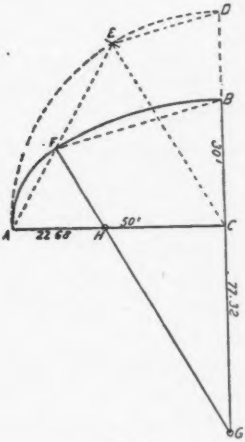
Fig. 8.

Take $CD = CA$. Draw AB and describe arc AD from center C.

Take $BE = BD$. Draw GF perpendicular to AE bisected in G, cutting AC in I, then F and I are centers required.

Draw the curve as before.

Construction No. 3. By Mahan. Let the angles at H and G be 60° and 30°.



Take $CD = CA$. Draw quadrant, DEA.

Take $AE = AC$. Draw DE.

Draw BF parallel to DE. Draw CE.

Draw FG parallel to EC, cutting AC in H and BC in G.

Then G and H are centers required.

Draw the curve as before.

Fig. 9.

Construction No. 4. By Würtele. Let the angles at H and I be 60° and 30°.

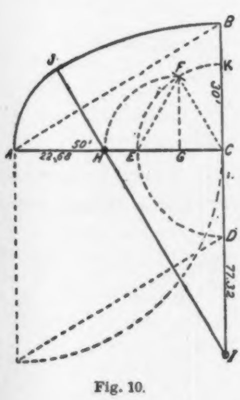


Fig. 10.

Take $BD = AC$.

Take $CE = CD$. Draw quadrant EFK.

Take $EF = CE$. Draw FC.

Draw FG parallel to BC.

Take $GH = GF$.

Draw HI parallel to FC and produce to J, cutting BC in I.

Then I and H are centers required.

Draw the curve as before.

Construction No. 5. From Haskell. Take angles at L and H at 60° and 30°.

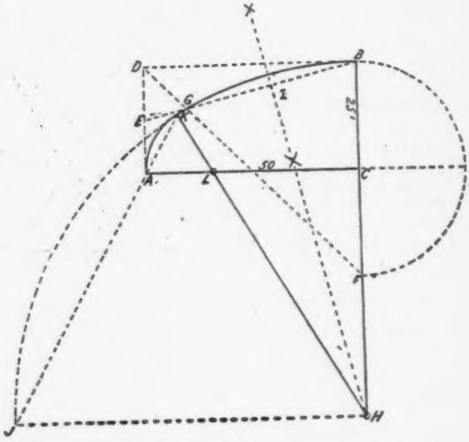
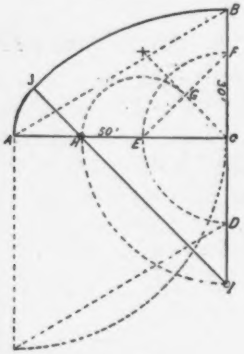


Fig. 11.

Take $BD = CA$ and parallel to CA. Take $AD = CB$ and parallel to CB. Take $DE = EA$. Draw EGB. Take $CF = CB$. Draw FD. FD cuts BE in G. Draw HI perpendicular to GB bisected in I. Draw $HJ = HB$ parallel to CA. Draw quadrant $BGKJ$. Draw JA cutting arc BG in K. Draw HK cutting AC in L. Then H and L are centers required. Draw the curve as before.

Construction No. 6. By Würtele. Let the angles be 45°.



Take $BD = AC$.

Take CE and $CF = CD$. Draw EF.

Draw CG perpendicular to EF, cutting EF in G.

Take $EH = EG$.

Take $CI = CH$. Draw IHJ.

Then I and H are centers required.

Draw the curve as before.

Fig. 12.

Construction No. 7. General construction.

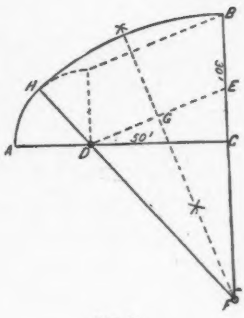


Fig. 13.

Take $AD = BE$, less than BC and strike curve AH. Draw DE.

Draw FG perpendicular to DE, bisected in G.

Draw FD to H.

FD are centers required.

Draw the curve as before.

Construction No. 8. From Nystrom. Let the angles be 30° and 60°.

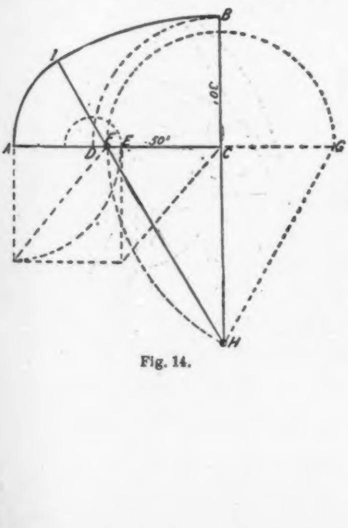


Fig. 14.

Take $CD = CB$.

Take $DE = \frac{1}{3} AD$.

Take $CF = AE$.

Take $CG = CF$.

From G as a center, with radius GF, strike an arc, cutting BC, produced in H.

Draw HF to I.

Then H and F are centers required.

Draw the curves as before.

Construction No. 9. Old construction.

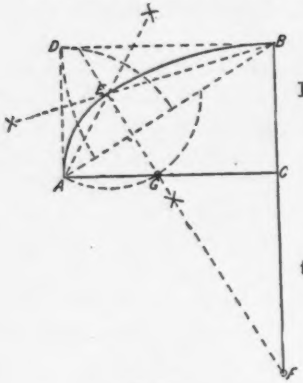


Fig. 15.

Draw BD parallel and equal to CA.
Draw AB.
Draw $AD = CB$.
Draw BE, bisecting angle ABD.
Draw AE, bisecting angle BAD.
BE cuts AE in E.
Draw EF perpendicular to AB, cutting AC in G.
Then F and G are centers required.
Draw the curves as before.

Five-centered Curves.

Construction No. 1. By Würtele. Let the angles be 30°.

Take $CF = CA$ on CB produced.
Draw quadrant FA.
Take $AG = CA$.
Draw CD and $BD = CB$.
Draw GC, bisecting DB in E.
Take $BH = BF$.
Draw HI parallel to CA.
Take $HI = HB$.
Draw IJ parallel to GC, cutting AC in K.
Make $AL = LK$.
Take $JM = LK$.
Draw MN parallel to CG.
Draw OL parallel to DC, cutting MN in P.
MPL are centers required.

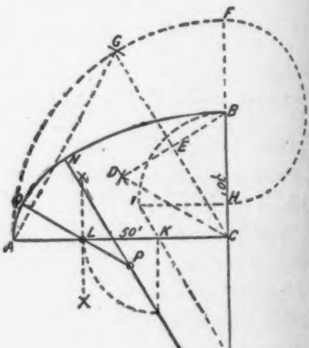


Fig. 16.

With radius MB at center M, draw arc BN. With radius PN at center P, draw arc NO. With radius LO at center L, draw arc OA.

Construction No. 2. From Johnson.

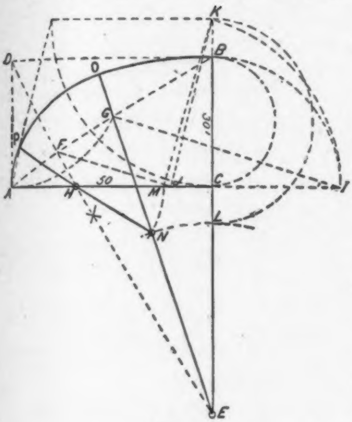


Fig. 17.

Draw BD parallel to and $=$ CA.

Draw $AD = CB$. Draw BA.

Drop DE perpendicular to AB, cutting AC in H; cutting AB in F; cutting BC, produced in E.

Take $EG = AF$.

Take $CI = CB$ on AC produced. Draw GI.

Draw FJ parallel to GI.

Make $JK = JI$, cutting CB produced in K.

Take $CL = KB$.

Take $AM = KC$.

Strike arc LN from centre E. Strike arc MN from center H. Draw ENO and NHP. ENH are centers required. Draw the curves as in the last figure.

Construction No. 3. From Vose.

Draw BD parallel to and $= CA$.

Draw AD parallel to and $= CB$. Draw AB.

Draw DF perpendicular to AB, cutting AC in E and BC in F.

Take $AG = AD$.

Take $CH = CG$.

Draw FG produced to I.

Draw HE produced to K, cutting FG in J.

FJE are centers required.

Draw the curve as before.

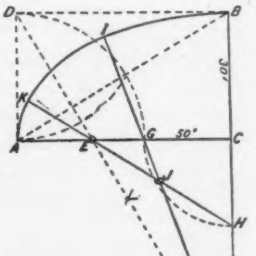


Fig. 18.

Construction No. 4. From Shaw.

Let $R = \dfrac{AC^2}{BC}$. Let $R_1 = \dfrac{BC^2}{AC}$.

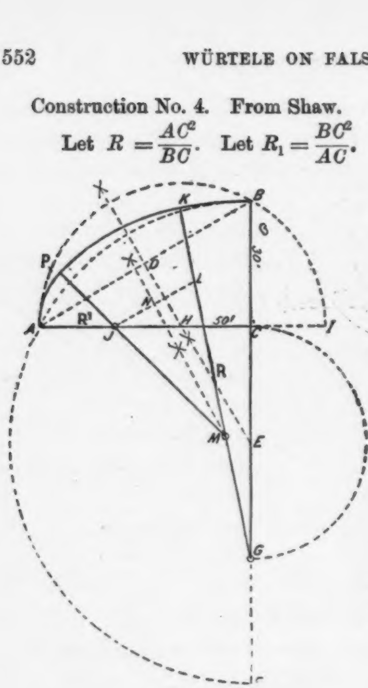


Fig. 19.

Draw AB.

Draw DE perpendicular to AB, and bisecting it in D and cutting BC in E and AC in H.

Take $EF = EB$ on BC produced.

Take $FG = BC$.

Take $HI = AH$ on AC produced.

Take $AJ = CI$.

Draw $GK = GB$ at any angle with GB.

Take $KL = AJ$. Draw JL.

Draw MN perpendicular to JL bisected in N, cutting GK in M.

Draw MJP.

GMJ are centers required.

Draw the curves as before.

Construction No. 5. From Mahan.

Take $BE = AD$ less than BE.

Draw DE.

Draw FG perpendicular to DE, bisected in F, cutting BC in G.

Bisect GE in H.

On center D with radius $DI = EH$. Draw arc IJ.

Take point J on arc IJ so that LM cuts DC.

Draw HJ.

Draw KL perpendicular to HJ, bisected in K.

Draw LJM. Draw JDN. LJD are centers required. Draw curve as before.

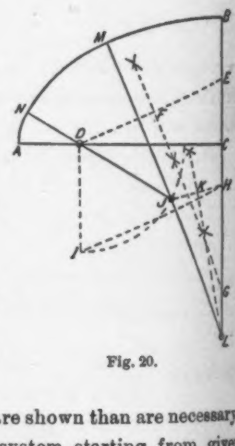


Fig. 20.

In above constructions many more lines are shown than are necessary, so as to indicate a continuous geometrical system starting from given

quantities and working only with parallel ruler and bow pen. Special constructions are obtained by reducing equations and plotting them geometrically, in which manner several of those given above were obtained.

Construction No. 6. From points on ellipse.

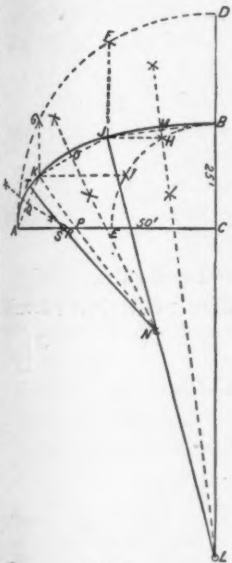


Fig. 21.

Draw quadrant *AD* to radius *CA*.

Trisect quadrant in *FG*.

Draw quadrant *BE* to radius *CB*.

Trisect quadrant in *HI*.

Draw *FJ* and *GK* parallel to *BC*.

Draw *HJ* and *IK* parallel to *AC*, thus obtaining points *J* and *K*.

Draw *ML* perpendicular to *BJ*, bisected in *M*.

Draw *ON* perpendicular to *JK*, bisected in *O*.

Draw *LJ*.

Draw *NK*, cutting *AC* in *P*.

Draw *QR* perpendicular to *AK*, bisected in *Q*, cutting *AC* in *R*.

Take $RS = \frac{1}{4} RP$.

Draw *NST*.

LNS are centers required.

Draw curve as before.

J and *K* are points on ellipse, and a multicenter curve is constructed by dividing quadrants into as many parts as required and proceeding as above.

When *BC* is over $\frac{2}{3}$ *AC* then *S* coincides with *P*.

Oval Curves.

Case 1.

Given greatest diameter.

Least radius $\frac{1}{3}$ diameter, and greatest radius $\frac{2}{3}$ diameter.

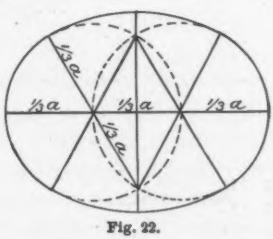


Fig. 22.

Case 2.
Given greatest diameter.
Least radius $\frac{1}{4}$ diameter.
Greatest radius $\frac{3}{4}$ diameter.

Fig. 23.

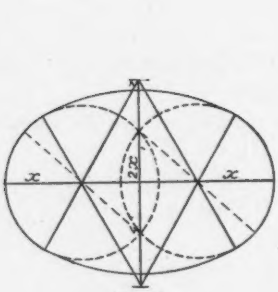


Fig. 24.

Case 3.
Given greatest diameter.
Least radius greater than $\frac{1}{4}$ diameter.

Egg-shaped Curves.

Case 1.
Given diameter and height.
Let $d = 0.77\ h$.
Radii $\frac{1}{2}\ d$, d and $\frac{3}{10}\ d$.

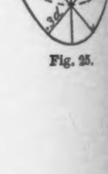


Fig. 25.

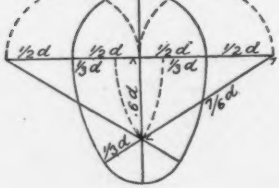


Fig. 26.

Case 2. Old sewers.
Let $d = \frac{7}{10}\ h$.
Radii $\frac{1}{2}\ d$, $\frac{3}{2}d$ and $\frac{1}{3}\ d$.

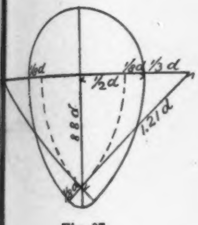


Fig. 27.

Case 3. Approved for sewers. Ency. Britt.
Let $d = \frac{2}{3}\, h$.
Radii $\frac{1}{2}\, d$, $\frac{4}{3}\, d$ and $\frac{1}{8}\, d$.

Case 4. From Molesworth.
Let $d = \frac{2}{3}\, h$.
Radii $\frac{1}{2}\, d$, $\frac{3}{2}\, d$ and $\frac{1}{4}\, d$, or $\frac{1}{3}\, h$, h and h.

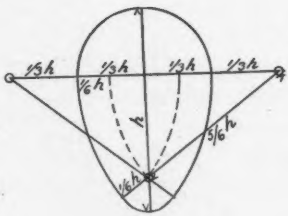


Fig. 28.

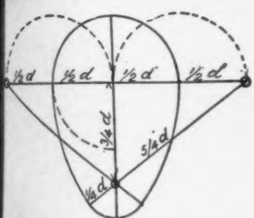


Fig. 29.

Case 5. From Merriman and Ency. Britt.
Let $d = \frac{2}{3}\, h$.
Radii $\frac{1}{2}\, d$, $\frac{3}{2}\, d$ and $\frac{1}{4} d$.

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482.

(Vol. XXIV.—June, 1891.)

NOTES ON A MOUNTAIN SLIDE.

By W. G. CURTIS, M. Am. Soc. C. E.

WITH DISCUSSION.

Land slips along railway lines are of such frequent occurrence and the methods employed for their quick removal are usually so simple and stand so completely within the category of commonplace experiences in railroad management, that, as a general rule, papers relating to such obstructions can possess little, if any, professional value. A mountain slide which obstructed one of the lines operated by the Southern Pacific Company in northern California last winter, occurred under rather unusual conditions; therefore a description of it and of the means employed to clear it away from the track may be of some interest. As indicated by appended topographical sketch, Fig. 1, tunnel No. 9 which is 670 feet long, is pierced through a spur on the easterly side of the Sacramento River cañon, and is approached from the north through 200 feet of "side hill" excavation in material which may be described as shaly talcose rock, with greatest depth of cut on center line ranging from 26

to 45 feet, and at the top of the slope on the mountain side something over 120 feet, the slope of cut averaging a little less than ½ to 1. Last season's rain-fall in this region was quite phenomenal, amounting to over 100 inches precipitation (rain and melted snow) for the five months ending March 1st, 1890, as against 59 inches average annual rainfall. As a consequence the ground was heavily saturated, and large masses of earth and slaty rock slid down from the mountain slope and fell upon the track, over the edge of a comparatively solid rock ledge which remained in place undisturbed by the slide above it, to a height of a little more than 100 feet above the railroad grade.

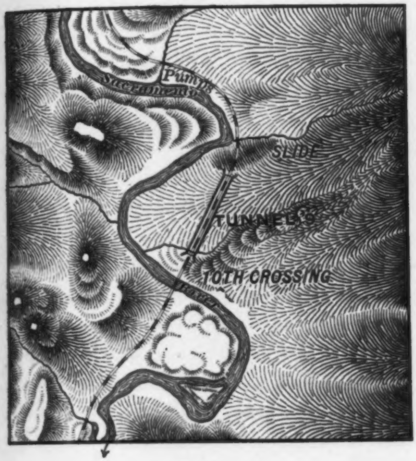

Fig. 1.

Plate No. LXIII gives a general view of the locality, and Fig. 2 is a reduced longitudinal profile; the length of slide measured along the track is about 200 feet, the height of rock slope is about 100 feet, and the vertical height from grade to top of slide about 300 feet. Roughly estimated, the total quantity of material

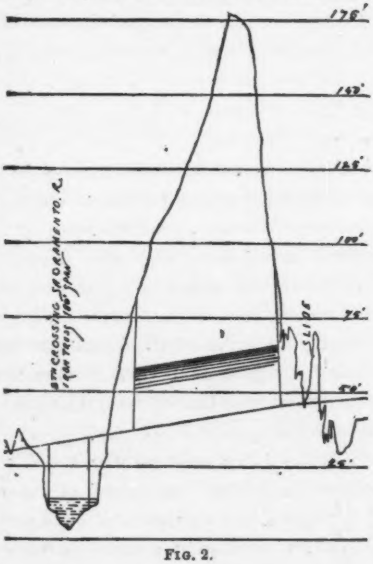

Fig. 2.

which, from first to last, fell upon the roadbed was 25 000 cubic yards. A road for team transfer of passengers, freight, etc., was immediately graded around tunnel No. 9 spur, and the work at the slide covered with all the men that could be handled, in day and night shifts. The weather being favorable, rapid progress was made, and when the track had been almost cleared with an expenditure of labor footing up 5 500 days work (the work of one man for eleven hours being taken as one day's work), a heavy storm of alternating rain and snow set in, the slide was again started and its movement continued until the pile resting upon the track had been restored to its original dimensions. Up to this time no serious difficulty had been experienced in working with carts and wheelbarrows all the men that could be placed upon the limited space occupied by the obstruction, but at this time the material around the edges of the slide had become so much softened up, that small or large masses of earth and rock were constantly dumping over the rock ledge and falling upon the workmen below, alarming them to such an extent that night-work became impracticable, and the day men were soon so much endangered that under observance of all necessary precautions for their safety, very little effective work could be done.

At this juncture we decided to clear the track with appliances similar to those employed in hydraulic mining operations, which had been previously used for railroad work in the Sierra Nevada mountains. The regulation gravity head of water for this purpose could not, however, be quickly obtained, and the pumping plant illustrated in Plate LXIII was hastily designed to utilize machinery and materials already in our hands, or which could at once be obtained and made serviceable for other uses afterwards. Locomotives to supply steam were sent to the north end of the tunnel, and the other appliances delivered on cars at the south end and hauled on wagons and improvised sleds or stone boats from the south end of the tunnel to the nearest ground available for assembling the entire plant. The hydraulic stream was turned on the work, in forty-three hours from the time the first shipment of materials arrived at the south end of the tunnel, the snow falling heavily nearly all of this time.

Twelve ordinary standard "surface" steam pumps, with capacities ranging from 150 to 500 gallons of water discharged per minute at ordinary speed, and aggregating for the gang of pumps about 3 300 gallons discharging capacity per minute, were placed end to end on a bed-

frame 32 inches wide and 128 feet long, constructed with railroad ties and trestle stringer timber. The line of pumps was set about 100 feet from the river bank and about 15 feet above the surface of the water in the river. All of the pumps were connected to a 12-inch, lap-welded pipe, and a suction pipe was laid from each pump to the river. The southerly end of the 12-inch discharge pipe was connected to a circular air chamber, 60 inches in diameter and 96 inches high, which served admirably to equalize the fluctuations of the pumps, giving the stream practically the same steadiness as the ordinary gravity pressure jet of the hydraulic miners. From the air chamber to the face of the work, water was conducted through pipe made of No. 12 Birmingham gauge, sheet iron, and riveted up in the shop into sections 30 feet long, one end of each section being flared out and the other having its diameter slightly reduced. The pipe was connected up on the ground by inserting the small end of one section of pipe into the larger end of another and driving it in to a snug fit, or making, in mining parlance, a tight "stove-pipe joint." A standard hydraulic giant butt was used, part of the time with a 4-inch discharge nozzle, and at other times with a 3-inch nozzle, according to nature of the material to be washed. The material was washed down on a flat place bordering the river. The method of working is quite plainly shown by Plate No. LXIV; the upper end of the flume and "head dams" being merely planks set on edge to guide the water-borne material or tailings into the flume conducting it away from the work. A small force of men were kept at work to break up, with sledges or by the use of powder, the larger pieces of rock and masses of snow and earth frozen together, and to otherwise expedite the work.

The quantity of material removed by the water jet for nine days (the time taken to open the line), aided by the laborers as above, may be estimated at 9 000 cubic yards, equal to, say, an even 1 000 cubic yards moved each twenty-four hours. Allowing for the time shut down, pumps cut out for repairs, etc., the average discharge of the water jet, under a pressure of 45 to 50 pounds per square inch, may be set down at 2 000 gallons per minute or 200 miner's inches a day. Each miner's inch moved 5 cubic yards each twenty-four hours, a somewhat higher duty than is ordinarily shown in hydraulic mining operations, but this might fairly be expected in handling material previously loosened up, as in the present case.

The average cost per day (twenty-four hours) of operating the hydraulic plant is closely estimated, as follows:

Wood, 25 cords	$75 00
Eight firemen and pumpers	20 00
Machinists and pump repairers	25 00
Men operating hydraulic jet (experts, paid high rates)	30 00
Thirty laborers	50 00
Total	$200 00

or, 20 cents per cubic yard.

Since the track was cleared, protection work (indicated by the appended cross-section plan, see Fig. 3) in the form of shedding, was

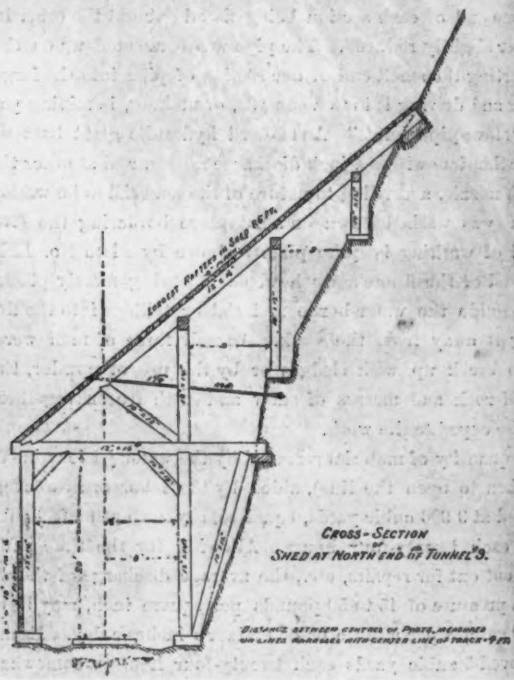

FIG. 3.

extended northward from the tunnel portal about 100 feet. This structure has proved to be sufficiently strong to divert earth and rock which since its construction has fallen down from the slide.

PLATE LXIII.
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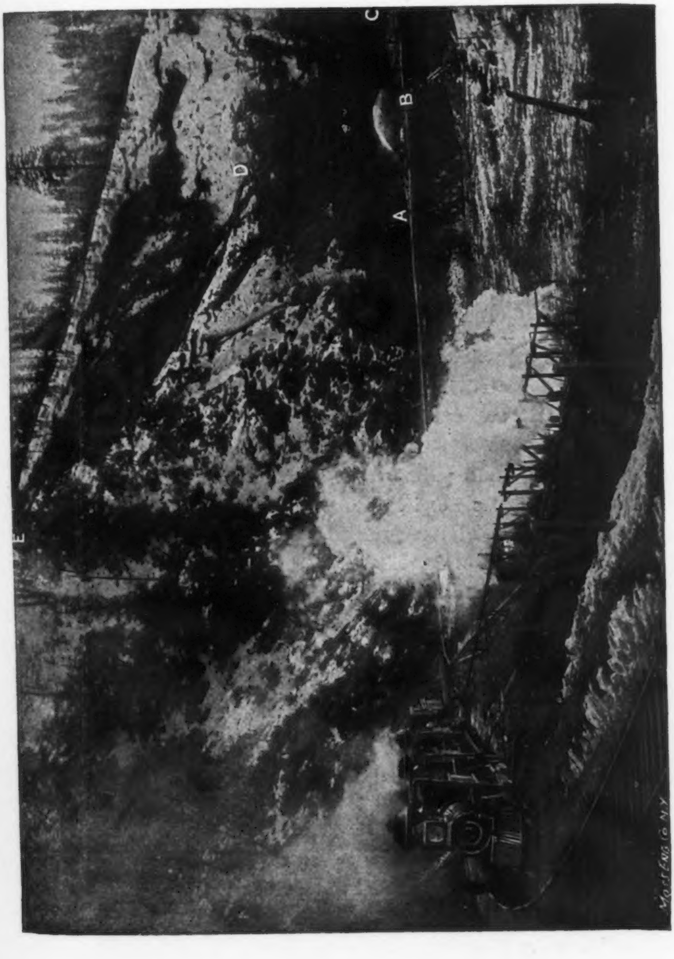

PLATE LXIV.
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CURTIS ON MOUNTAIN SLIDE.

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The appearance of the pile of material covering the roadbed near the completion of the work is quite clearly indicated by the Plates. The top of this pile of material, as it originally came down, was 12 or 15 feet above the top of the tunnel arching, filling the northerly 200 feet of the tunnel with earth to an average depth of about 10 feet. As soon as a "bench" had been taken out with wheelbarrows, low enough down to open an air hole into the tunnel, the removal of earth was carried on within it on cars loaded by the use of wheelbarrows.

DISCUSSION.

William P. Shinn, President Am. Soc. C. E.—The use of the hydraulic jet to get rid of this slide was certainly unique and eminently successful, both as to the speed with which the work was done and as to the cost.

F. Collingwood, M. Am. Soc. C. E.—When preparations were making for sinking the Brooklyn cassion of the East River Bridge I was directed to make some experiments to determine the quantity of sand which could be transported by a jet or small stream of water. The intent was that in case they proved promising the material excavated in the chambers should be carried by this means to the water shafts. The experiments had only progressed far enough to show the practicability of the scheme when it was decided for special reasons to abandon it. In making the experiments a cubical box was made holding a cubic yard, and a trough arranged so that its grade could be readily changed. Water being admitted into the trough, sand was shoveled in at the same time almost as rapidly as the stream would carry it away. It was found that any inclination less than about one in ten left a deposit, and that with that slope, the volume of sand carried was by actual measurement about 50 per cent. of the volume of the water. An experiment was made with a 4-inch siphon pipe, the long leg about 1 foot in excess of the other. This carried the sand faster than two men could possibly shovel it. The tabular results obtained were given in a short paper to the Society in the first volume of the *Transactions*.

Charles B. Brush, Director Am. Soc. C. E.—In water works in the western States, especially along the Missouri River the amount of sediment is very great. In the first settling basins the sediment will amount to a depth of 10 feet in one year. This sediment is removed usually by a water jet from a 2½-inch hose. The drain from the basin is about 2 feet in diameter, if the size is greater the drain is very apt to choke up. If the sediment is allowed to remain in the basins

until the accumulation is 6 or 8 feet deep there is considerable difficulty in removing it with the water jet. On a depth of 2 feet the jet acts with little difficulty, for greater depths the material becomes so compact that the jet will simply make a hole in the silt without removing it. If the basin is cleaned when the silt is not over 2 feet in depth the water jet is very effective. The bottoms of the basins are inclined about one in ten.

Mr. COLLINGWOOD.—What proportion of water do they have to use to the amount of silt removed?

Mr. BRUSH.—I am not definitely informed on that point, the amount depends very much on the depth of the silt.

J. FOSTER CROWELL, M. Am. Soc. C. E.—Is the difference observed due to the difference in quantity or quality?

Mr. BRUSH.—It is due to quantity, to the weight of the silt that has accumulated. The upper 2 feet are easily removed but further down the silt is very tenacious.

Mr. CROWELL.—I have had no experience of my own, but some time ago I investigated the results of a jet operation in Georgia, and found that the quality of the material to be removed had a very important effect in the economy, as might be supposed. With loam and with sandy soils generally the economy was very great, the quantity removed was very much in proportion to the water, but as the clay predominated the economy became less and less, and there are clays which cannot be removed at all by the water jet. I should suppose from the statement of the cost of this particular work of removing the slide, that the material was favorable for the purpose. Mr. Curtis estimates the cost as about twenty cents per yard; of course we must make allowances for the conditions, the difficulty of applying the jet and other reasons; material is often removed for one-tenth of that, where the conditions are very favorable.

Mr. BRUSH.—In the work on the Hudson River tunnel the use of the water jet was attempted at the depth at which the tunnel is building, 50 feet below the bottom of the river. At that point the water jet had no effect at all, it simply would bore a hole. The water jet on the silt at the surface is very effective; the only way we could use the jet in the Hudson River tunnel was to break up the silt in small pieces and then the water jet would work easily.

P. F. BRENDLINGER, M. Am. Soc. C. E.—I am sorry the author of the paper is not here, because I would like to know whether he could not have applied a steam shovel to greater economy than a water jet. It may be that he had no steam shovel, which would be a good excuse, but I feel satisfied that a steam shovel would have done the work at at least half the cost. It is pretty difficult to make a contract to take out material with a steam shovel at ten cents a yard, but when the company has a steam shovel they can do it much cheaper. Twenty cents is certainly a very

high price for moving a silt of that nature. A good steam shovel would certainly move more than a thousand yards in twenty-four hours, it ought to move 1 500, the conditions being good and the cars being promptly furnished; time is very frequently an object, and I should say it would have been in this particular case.

J. H. Wallace, M. Am. Soc. C. E.—Being familiar with the work of moving this slide, as described by Mr. Curtis, I will say, in answer to Mr. Brendlinger's query as to whether a steam shovel could not have been used to greater economy than a water-jet, that although the company had a steam shovel which was within reach at the time, still the situation was such that a steam shovel could not be used. The slide was in the mouth of a tunnel, in a very mountainous country, and it would have been a very expensive matter to lay any track on which to place cars to be loaded by the shovel. It would then have been necessary to excavate with the steam shovel a cut wide enough not only for the operation of the steam shovel, but also for a track on which to stand cars while being loaded. As such a track would have abutted against the material of the slide, it would have been necessary to make a switch every time a car was loaded, removing the loaded car and placing an empty one in position. Under these circumstances, a steam shovel could not have handled more than four or five hundred yards a day. Moreover, it was extremely doubtful whether a steam shovel could be kept at work. As noted by Mr. Curtis, after the track had been nearly opened by the use of carts and wheelbarrows, the slide started a second time and continued to move until the track was again completely covered, the material being piled up as high as when the first slide occurred. There was thus the danger that the slide might start again at any time and the shovel be completely buried. There was also danger from the large number of rocks of various sizes which were continually detached from the mountain above and precipitated into the cut. Some of these were large enough to have disabled the shovel, and all of them were large enough to have endangered the lives of the men employed on the shovel. While twenty cents per yard may be considered a high price for removing such material under most circumstances, in the case under discussion the object in view was to get the road open at the earliest possible moment and to accomplish this questions of economy were not considered as carefully as they are when work is being planned.